Arch Girder (Arched Frame)

نسألكم الدعاء

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اذا حملت تطبيق RC Structures YASSER EL LEATHY على تليفونك المحمول او اللوح السطحى
ستستطيع أن تشغل أفلام شرح للمقاطع التى تحتوى على رمز

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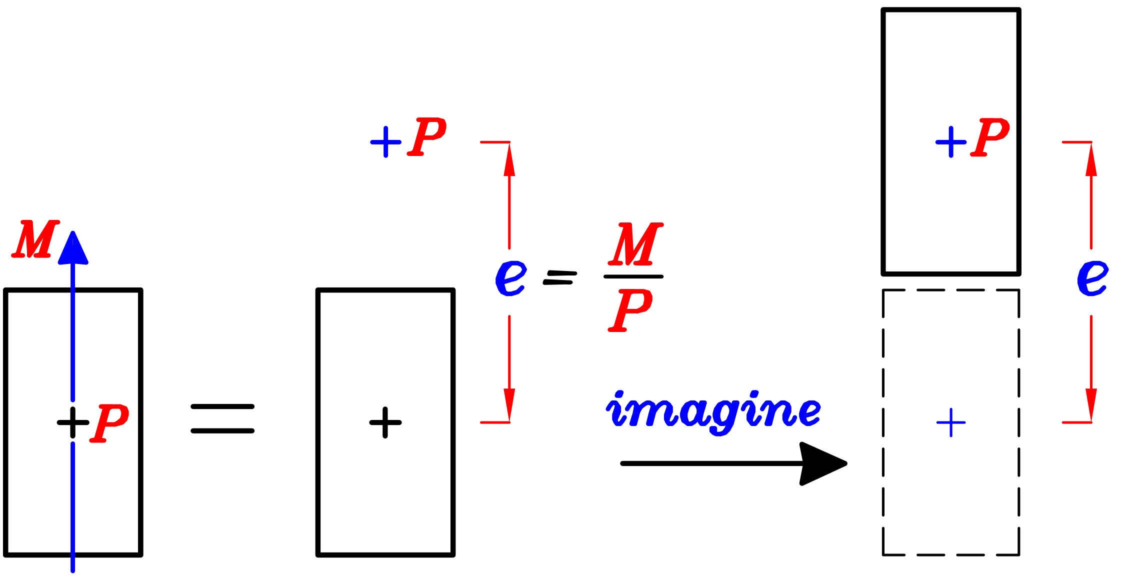
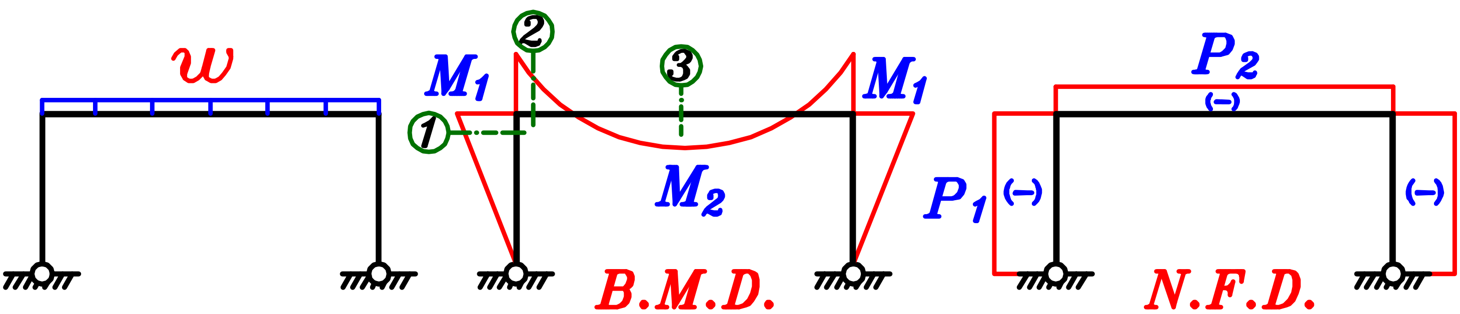
Introduction.

Thrust Line. (Pressure Line).

للقطاعات المؤثر عليها M , P اذا تخيلنا أنه تم ترحيل القطاع مسافه e عكس اتجاه الـ moment سيكون القطاع المرحل عليه Normal Force فقط وبالتالى عند تصميمه سيحتاج ابعاد قطاع اقل و كميه حديد تسليح اقل .



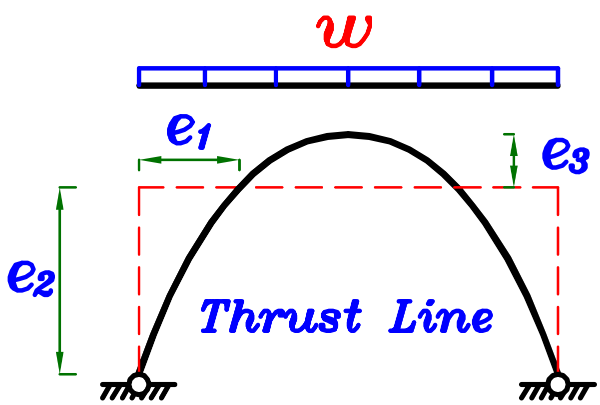
اذا استطعنا لاى structure ان نرحل كل قطاعاته عكس اتجاه الـ moment مسافه e سنضمن ان الـ structure الجديد كل قطاعاته سيؤثر عليها Normal Force فقط.
و بالتالى تكون ابعاد قطاعاته و كميات حديد تسليحه اقل فتكون تكلفته أقل .
و يسمى الـ structure الجديد Thrust Line أو Pressure Line .



Sec. 1 $\quad e_1 = \frac{M_1}{P_1}$

Sec. 2 $\quad e_2 = \frac{M_1}{P_2}$

Sec. 3 $\quad e_3 = \frac{M_2}{P_2}$



من المنشأت التى شكلها نفس شكل (*Thrust Line*) الـ *Arch Girder*

و لان فى هذه المنشأت تكون قيمه (*axial Force*) **تقريبا** ثابته على جميع القطاعات .

أى أن $\left(e = \frac{M}{P} = \frac{M}{constant} \right)$

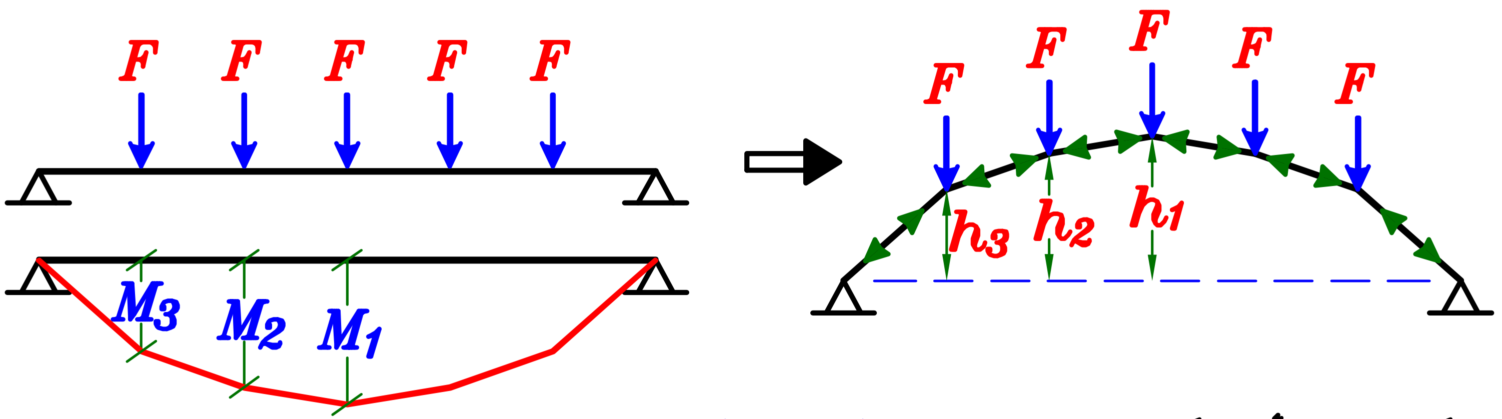
لذا اذا رسمنا شكل الـ (*structure*) عكس شكل الـ (*B.M.D.*) يكون هو نفسه

شكل الـ (*Thrust Line*) أى لا يكون عليه (*Bending moment*)

و لكن يؤثر عليه فقط (*axial Force*) .

و هذه تعتبر ميزه اقتصاديه لان هذا يوفر فى كميات كلا من الخرسانه و حديد التسليح .

الـ *Arch Girder* يحمل اكثر من *2 concentrated loads*



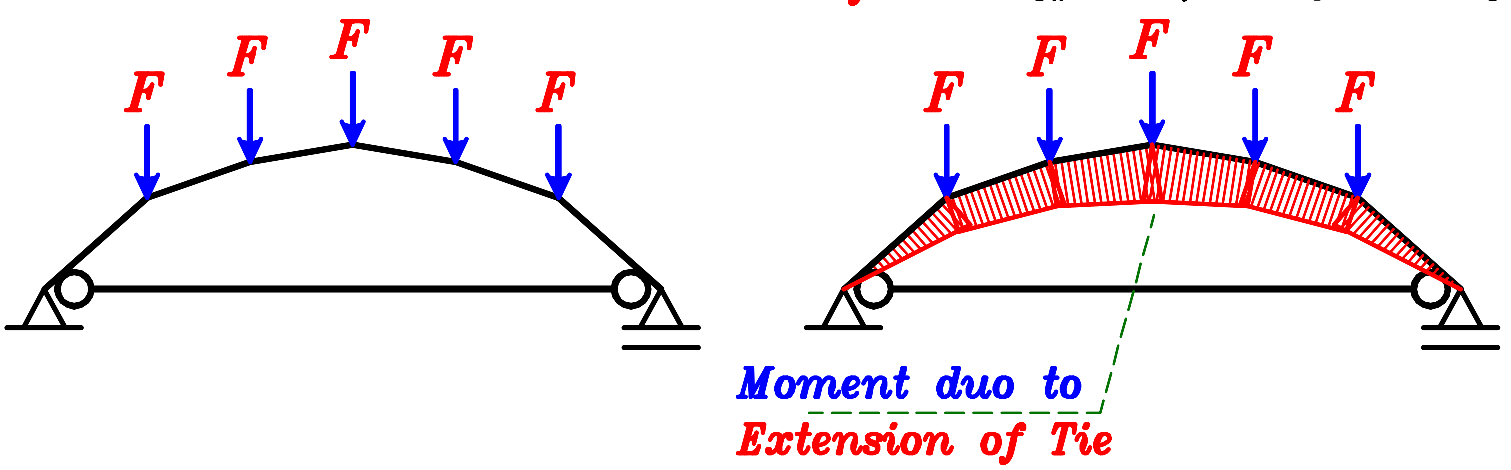
و لكى نضمن أن شكله عكس شكل الـ (*B.M.D.*) تماما

يجب أن تكون النسبه بين h_1 , h_2 , h_3 هى نفس النسبه بين M_1 , M_2 , M_3

و مثل الـ *Polygon Frames* من الصعب أن نأخذ *Hinged–Hinged Supports*

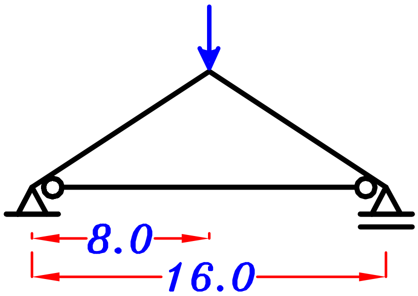
فسنضطر ان نأخذ *Hinged–Roller Supports* للـ *Arch Girder* مع وضع *Tie* بينهم .

و سنأخذ فى الحسابات تأثير الـ *Extension of Tie*



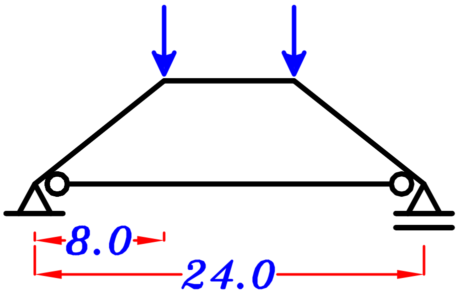
Number of Segments of Arch Girder.

2 Segments



Triangular Polygon Frame

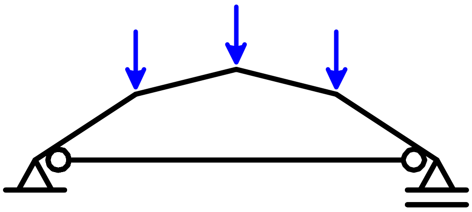
3 Segments



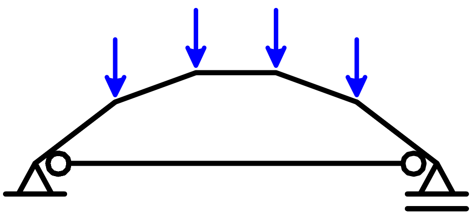
Trapezoidal Polygon Frame

IF the span is more than 24 m

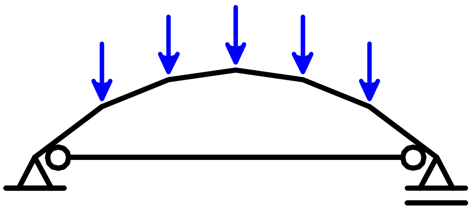
We will need more than 3 Segments, So we will need Arch Girder.



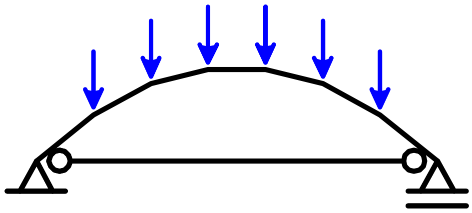
4 Segments



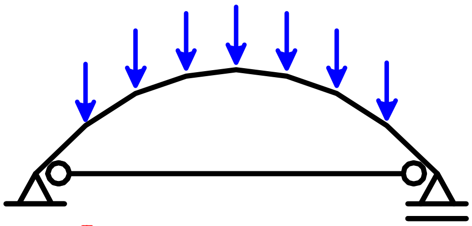
5 Segments



6 Segments



7 Segments



8 Segments

لتحديد عدد الـ Segments للـ Arch Girder يفضل :

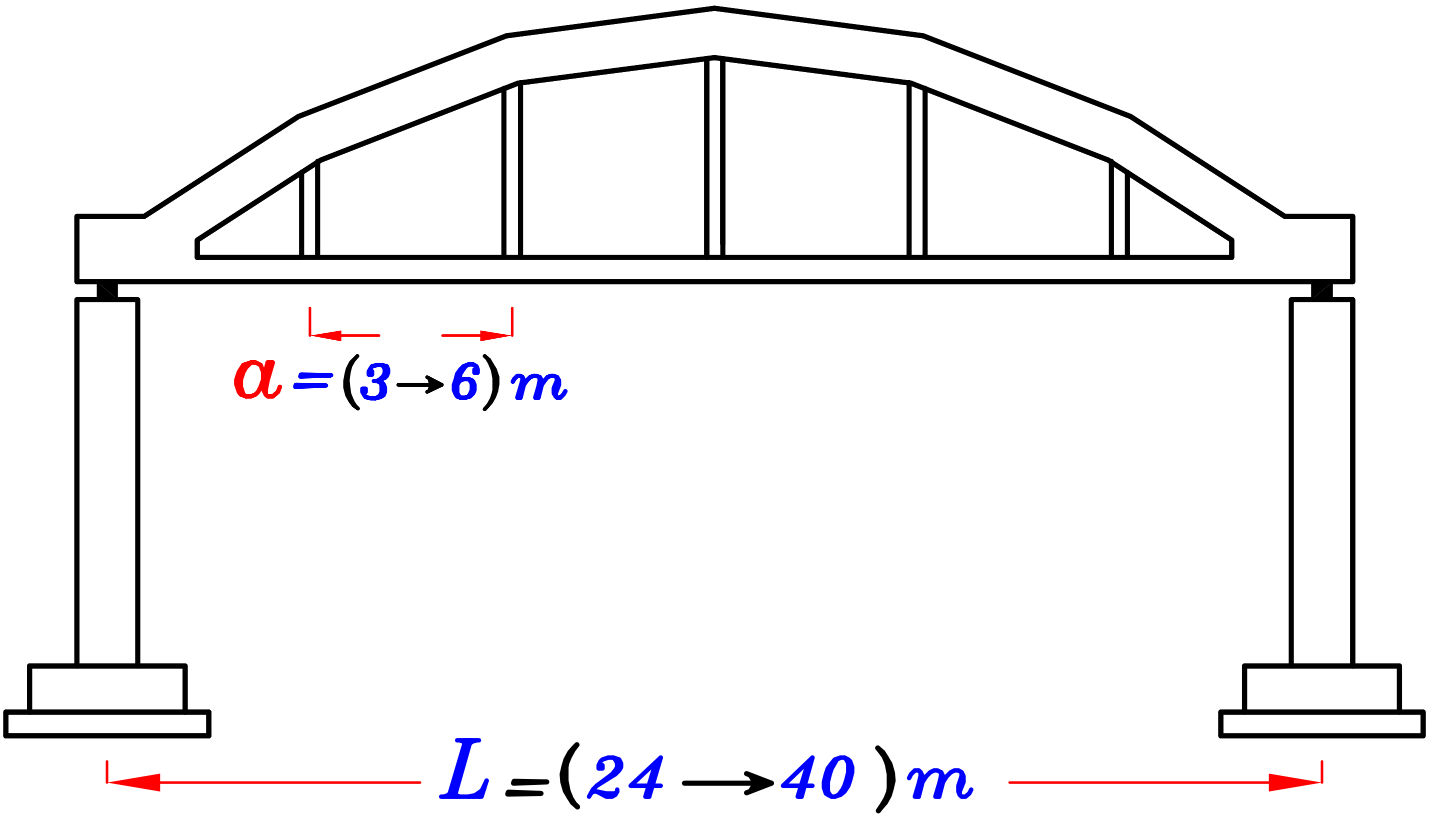
$a \ngtr 6.0\,m$

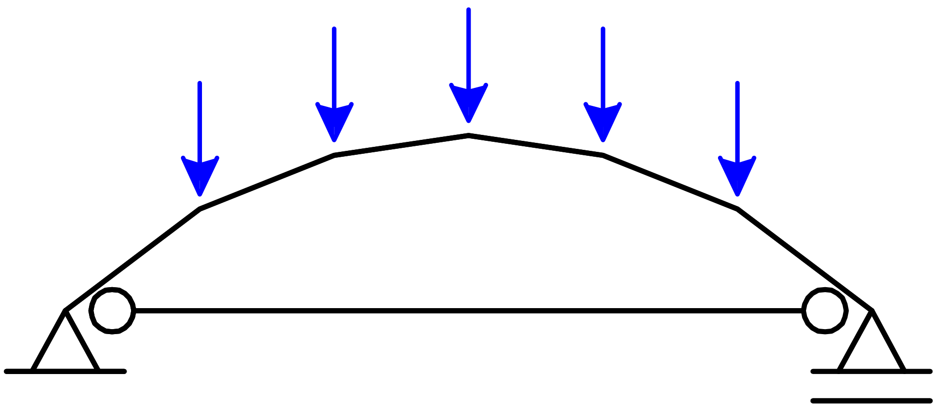
١ـ المسافه الافقيه بين الـ Joints لا تزيد عن ٦,٠ م

حتى لا تكون البلاطه الـ H.B. one way مكلفه

٢ـ يفضل (ليس شرط) أن تكون المسافات متساويه

حتى يكون شكله معماريا منتظم .





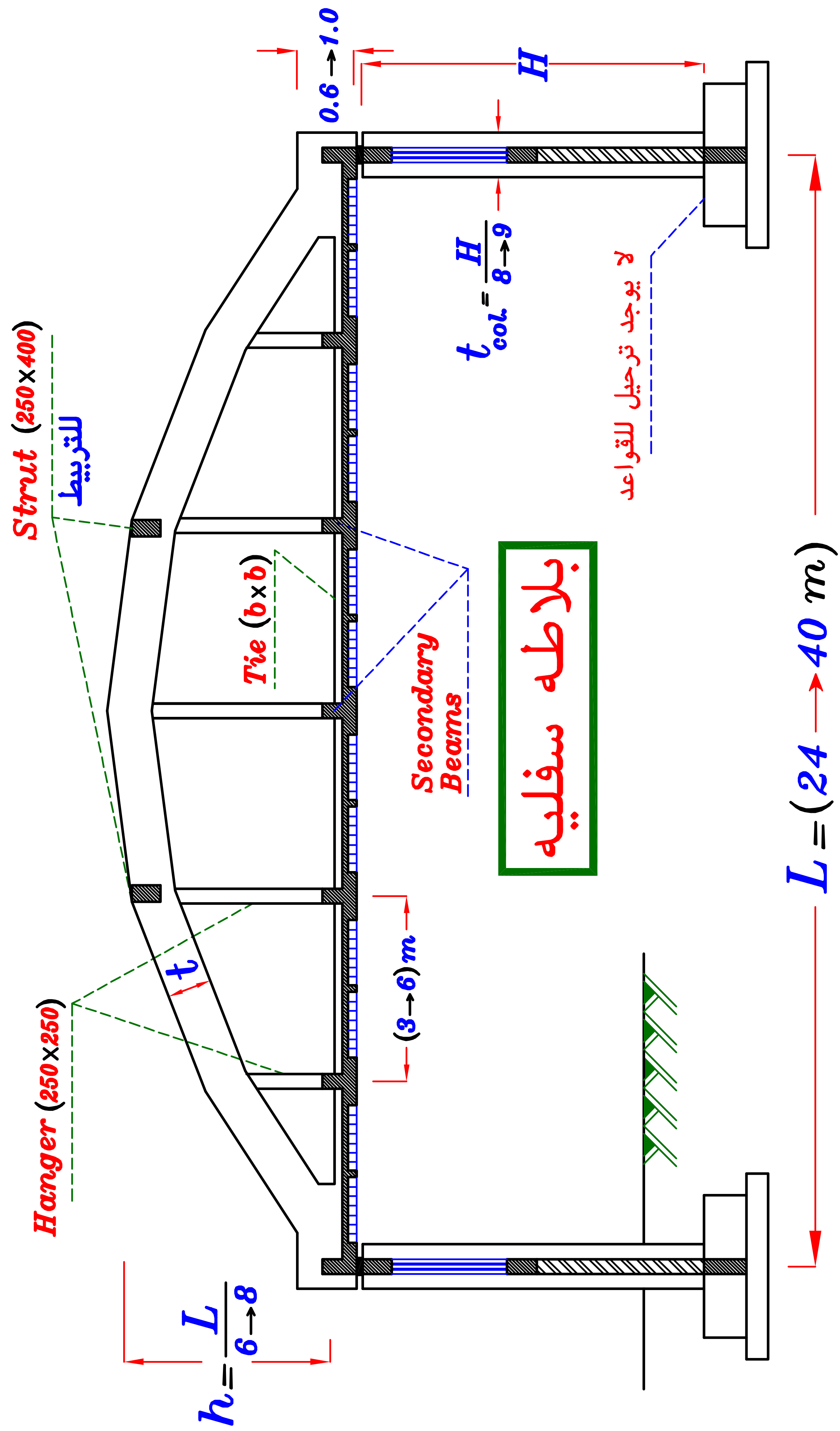
لكى نتحكم فى وجود أحمال مركزه عند ال Joints فقط :

١- نضع كل الكمرات المحموله على ال Arch عند ال Joints فقط .

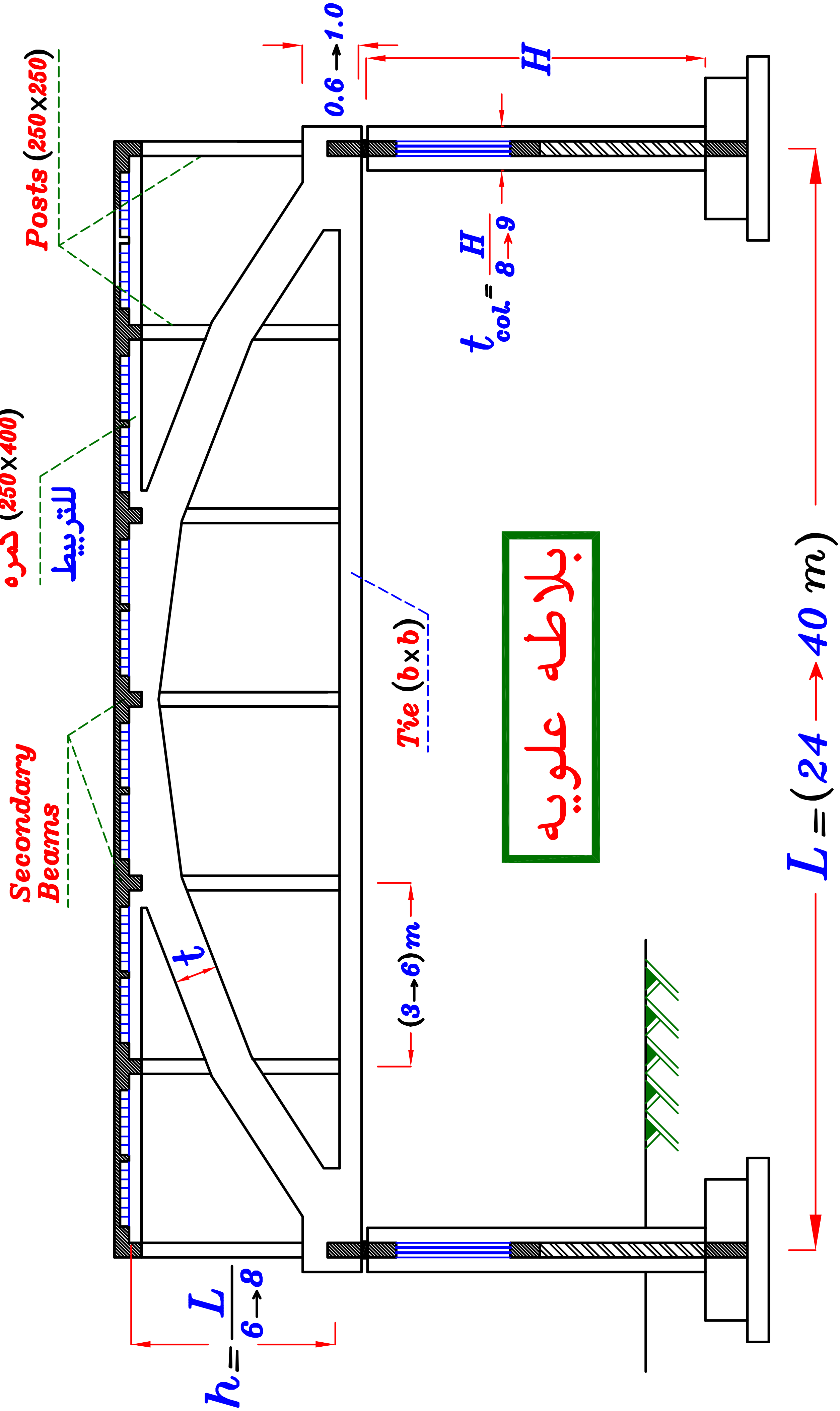
٢- نأخذ كل البلاطات One Way Slabs فى إتجاه الكمرات بحيث لا ترمى أى أحمال على ال Arch (تؤخذ One Way H.B. or One way Solid) .

٣- نضع أى post أو أى hanger عند ال Joints فقط .

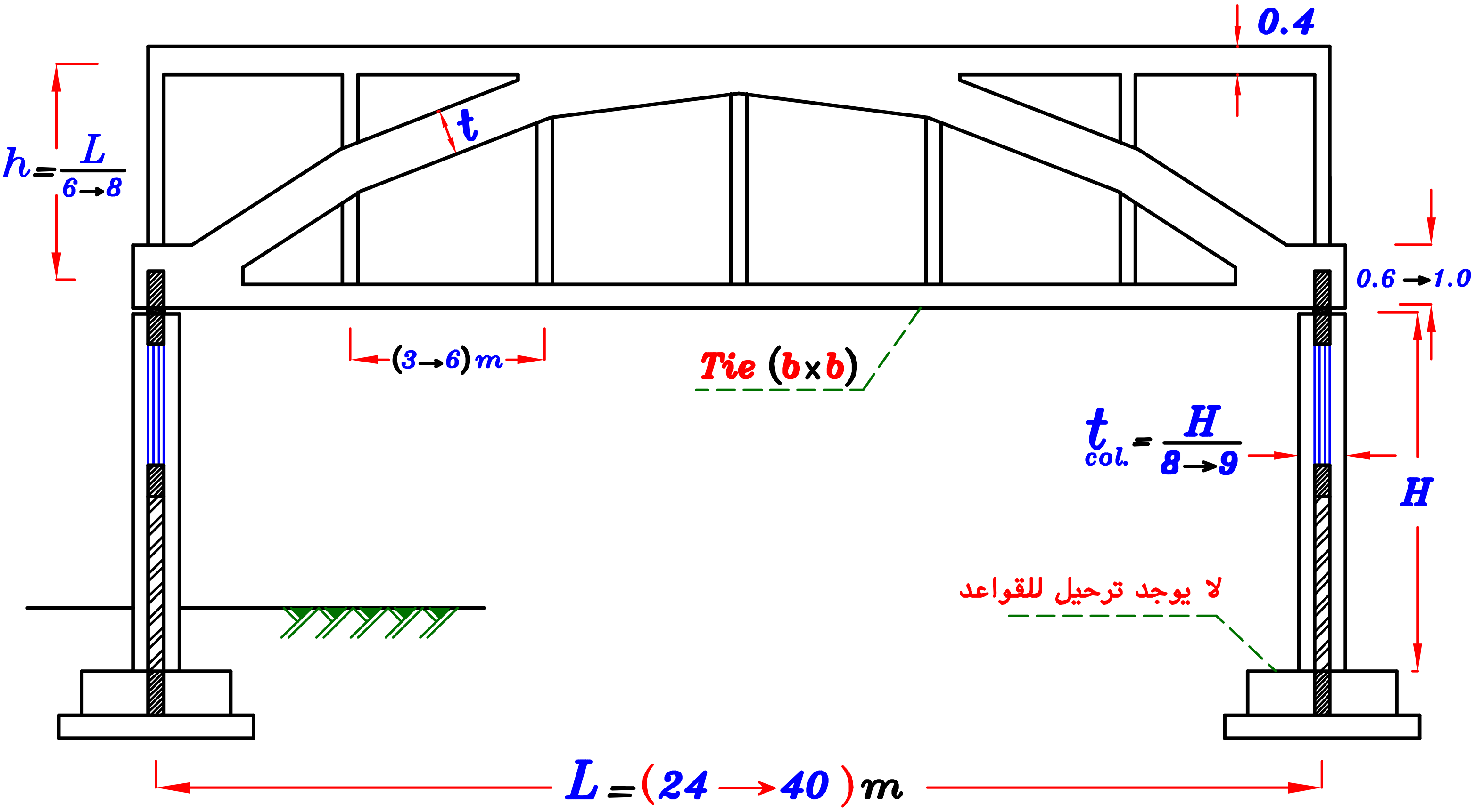
٤- نفرض أن ال O.W. لل Arch Girder يؤثر كأنه Concentrated Load عند ال Joints .



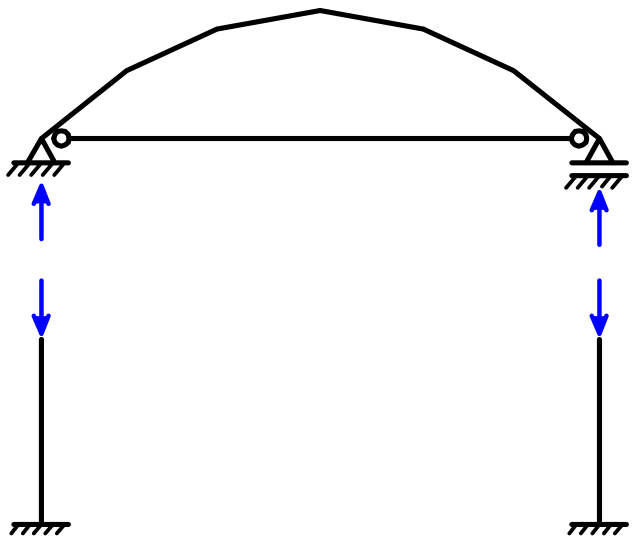
Strut (250×400)
للتربيط
Tie (b×b)
Secondary Beams
بلاطة سفلية
لا يوجد ترحيل للقواعد
H
0.6→1.0
$t_{col.} = \frac{H}{8\to 9}$
$L = (24 \to 40\ m)$
$h = \frac{L}{6\to 8}$
Hanger (250×250)
t
(3→6)m


Posts (250×250)
كمره (250×400)
للتربيط
Secondary Beams
Tie (b×b)
(3→6) m
L = (24 → 40 m)
h = L / (6→8)
t
H
0.6 → 1.0
t_col. = H / (8→9)
بلاطه علويه

Concrete Dimensions



* Span $(L) = (24 \rightarrow 40)\ m$
* Height $(h) = \dfrac{L}{6 \rightarrow 8}$
* $t_{(Arch)} \simeq \dfrac{L}{20 \rightarrow 25}$
* Tie $(b \times b)$
* Hanger (250×250)
* $t_{col.} = \dfrac{H}{8 \rightarrow 9}$



Statical System

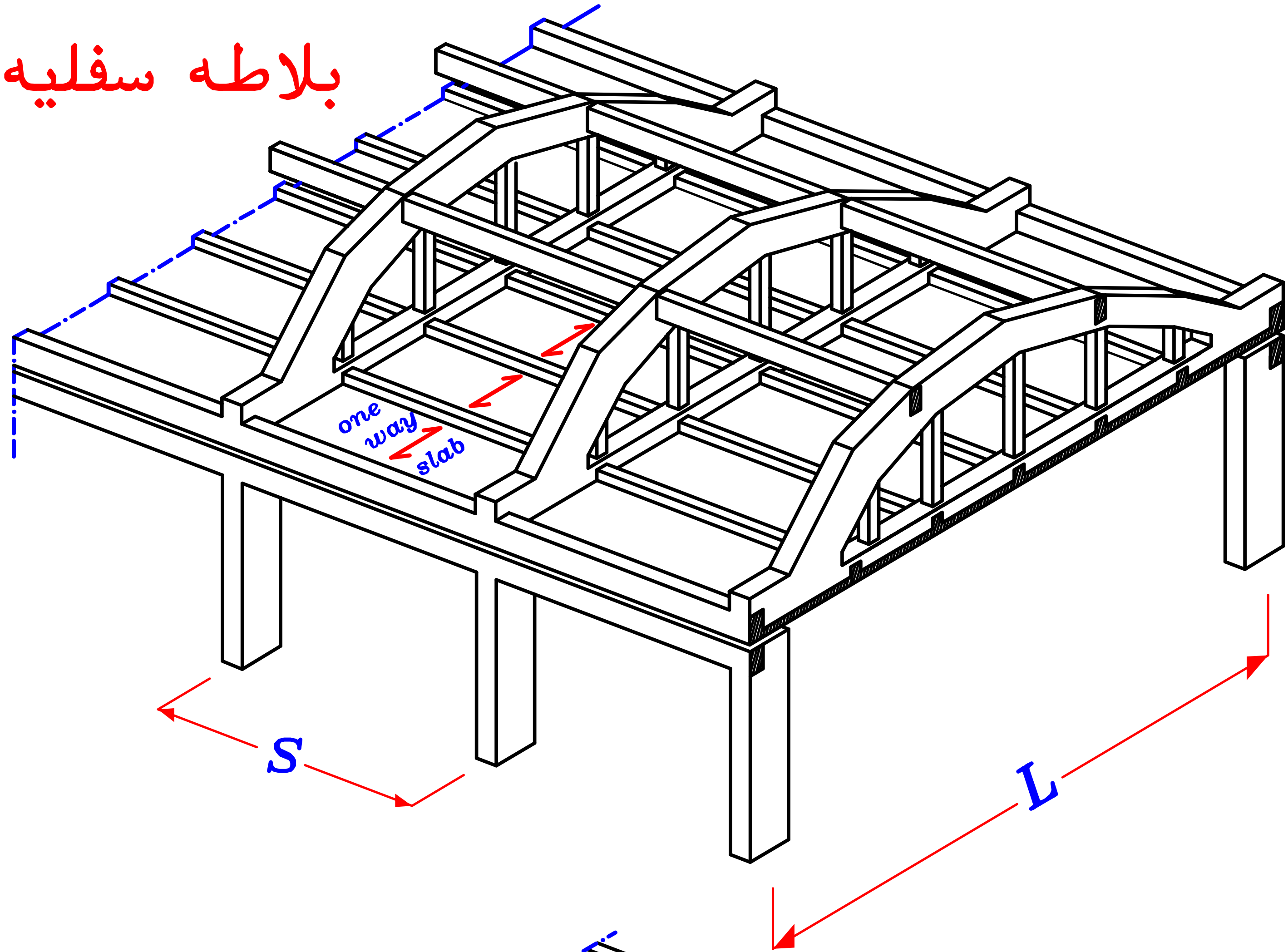
يجب أن تكون كل الأحمال مركزه عند ال Joints فقط و لكى نتحكم فى ذلك يجب أن :

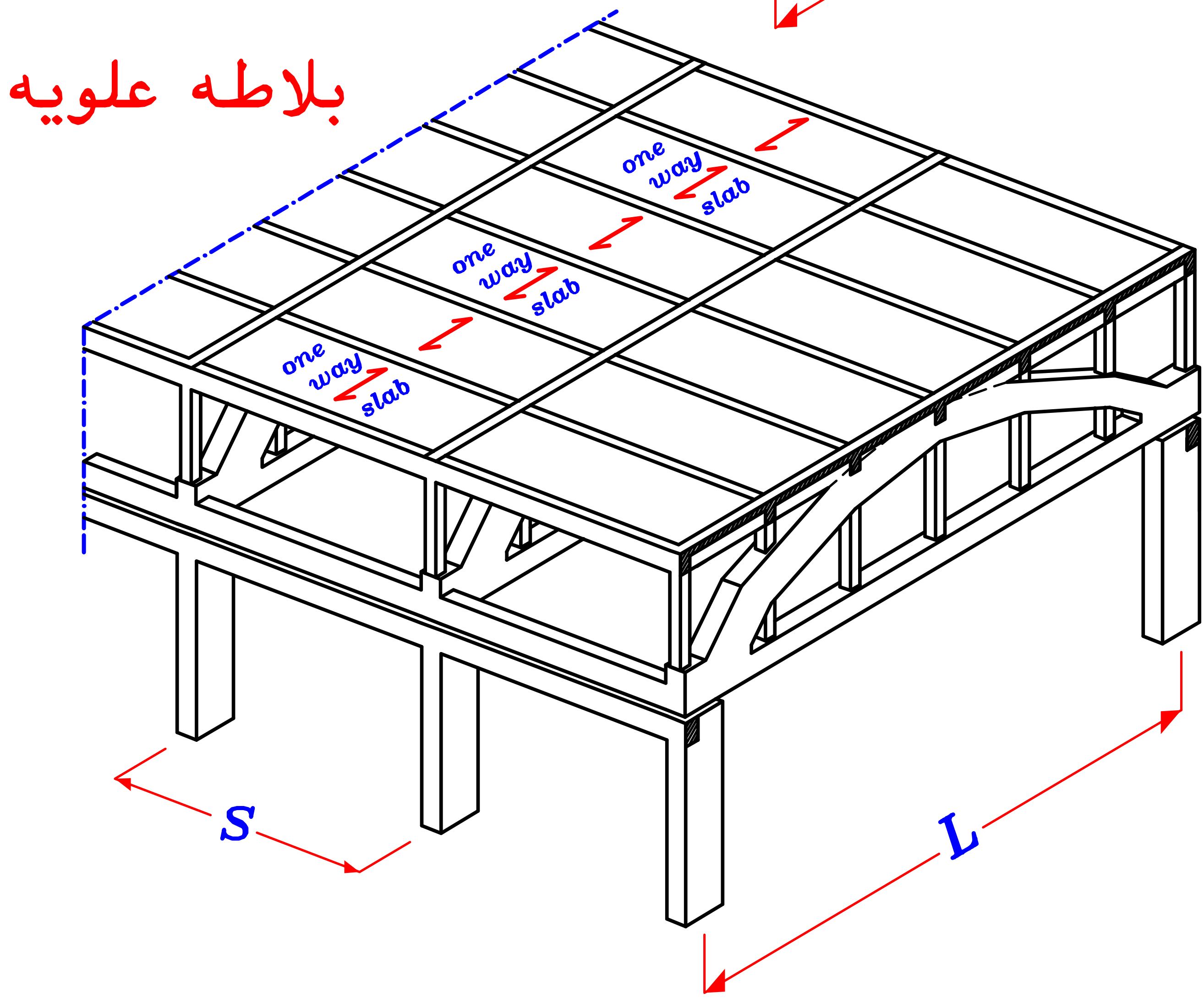
١- نضع كل الكمرات المحموله على ال Girder عند ال Joints فقط .

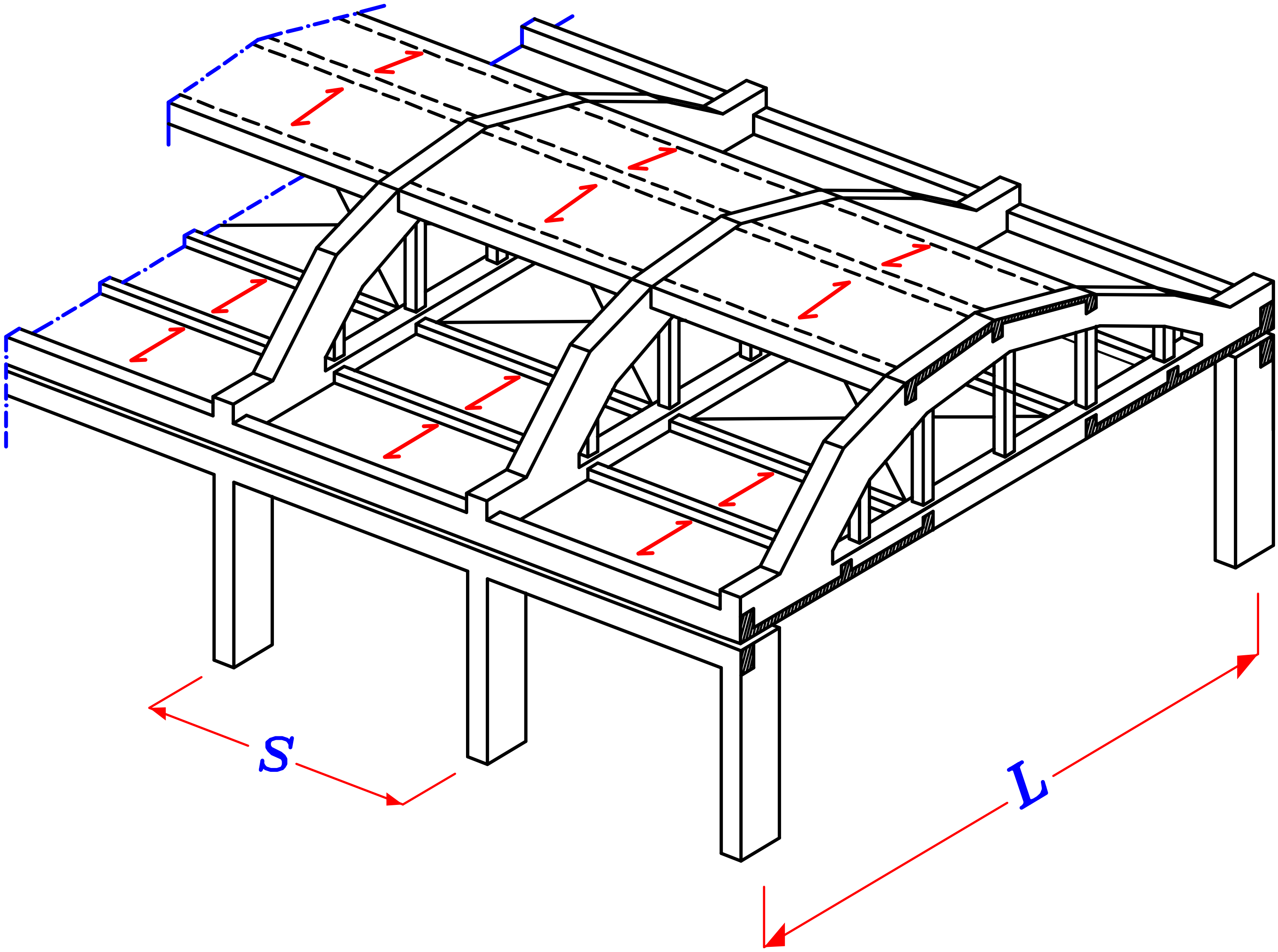
٢- نأخذ كل البلاطات One Way Slabs فى إتجاه الكمرات بحيث لا ترمى أى أحمال على ال Girder .

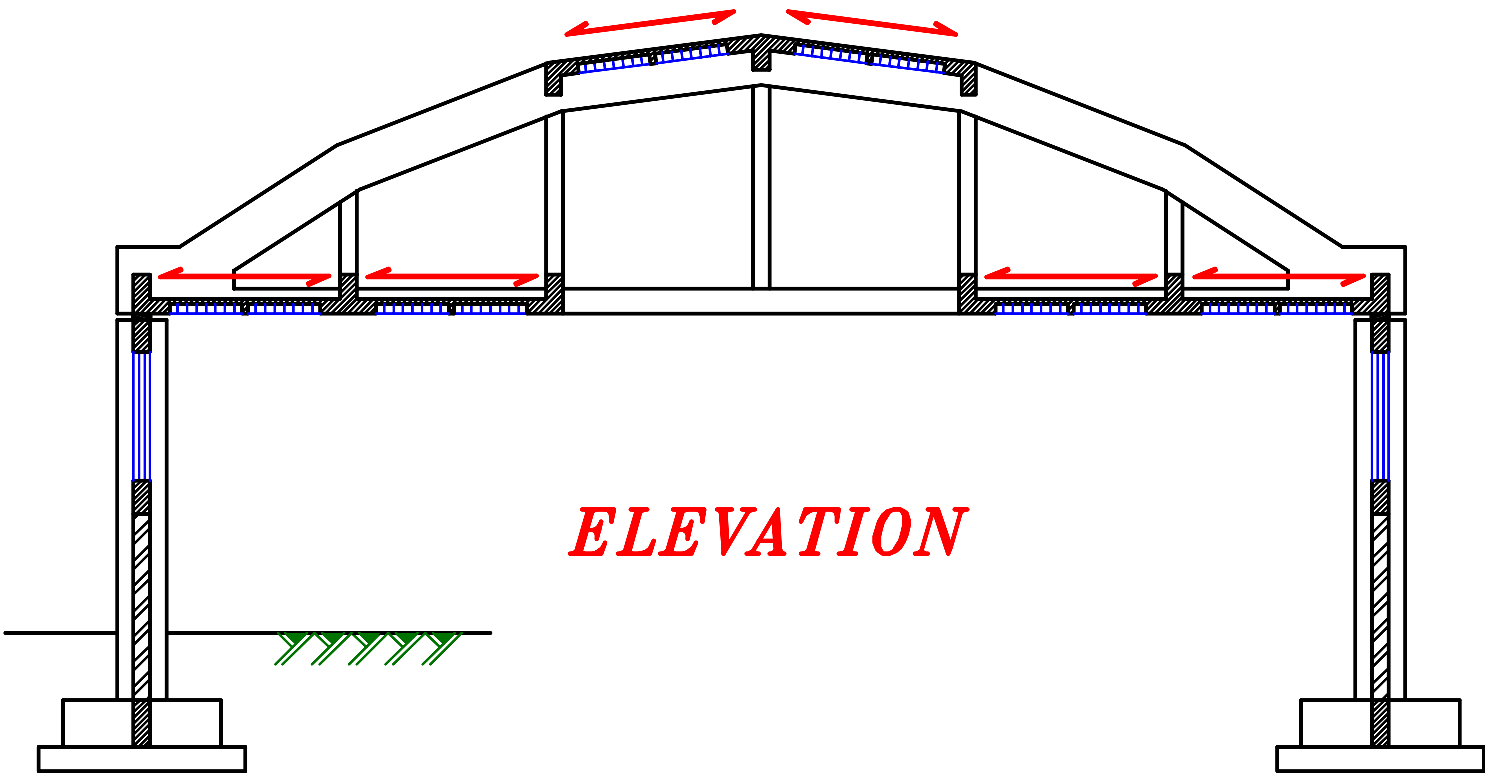
٣- نضع أى post أو أى hanger عند ال Joints فقط .

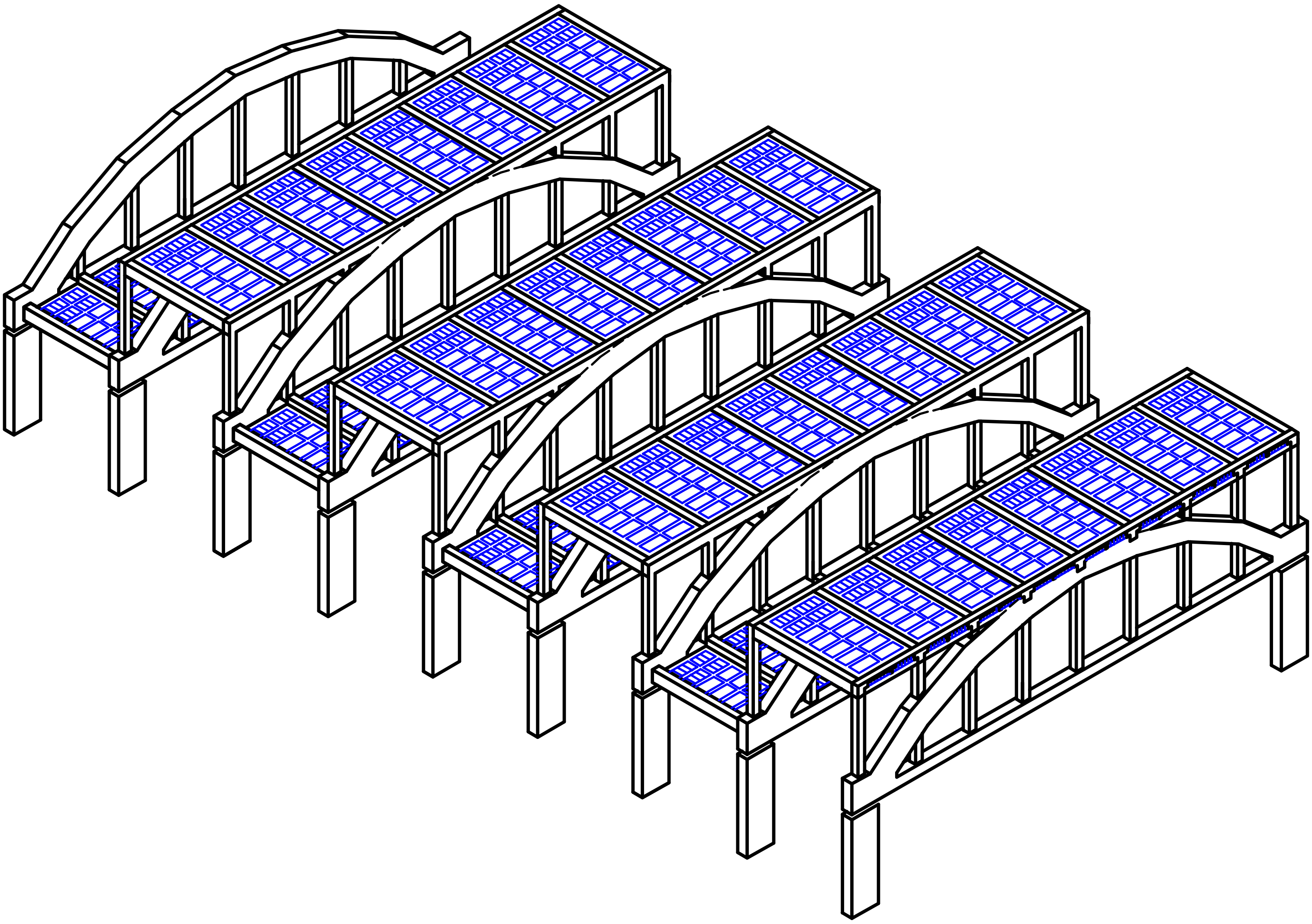
٤- نفرض أن ال O.W. لل Girder يؤثر كأنه Concentrated Load عند ال Joints.

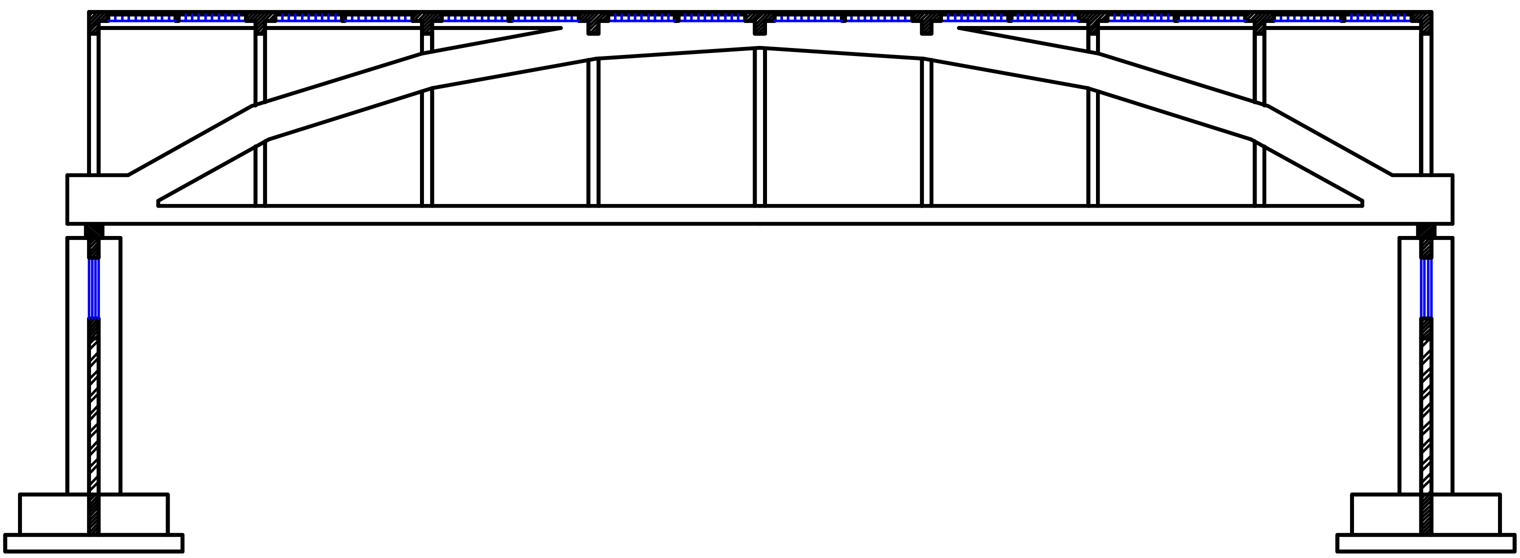

بلاطه سفليه
one way slab
S
L

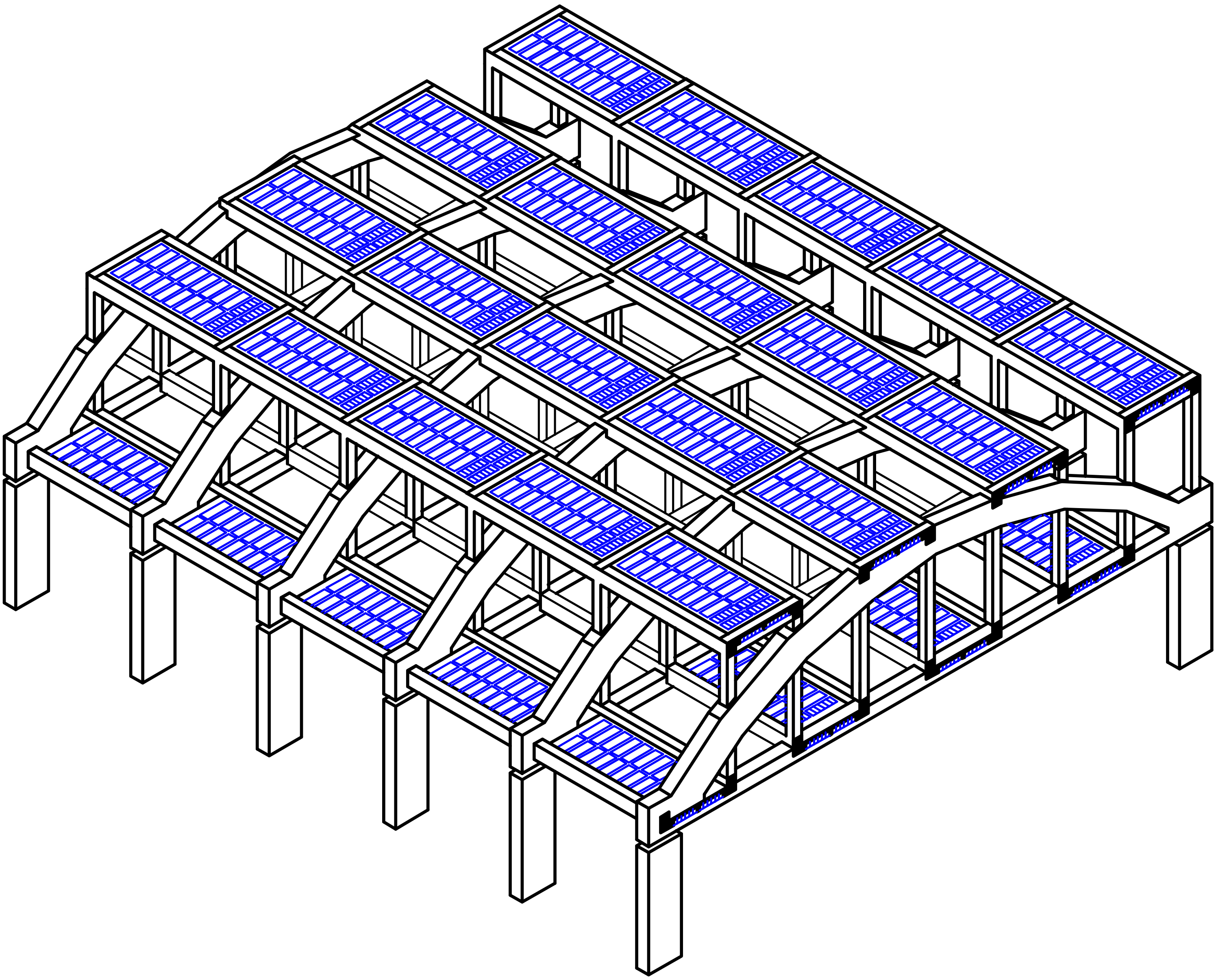

بلاطه علويه
one way slab
one way slab
one way slab
S
L

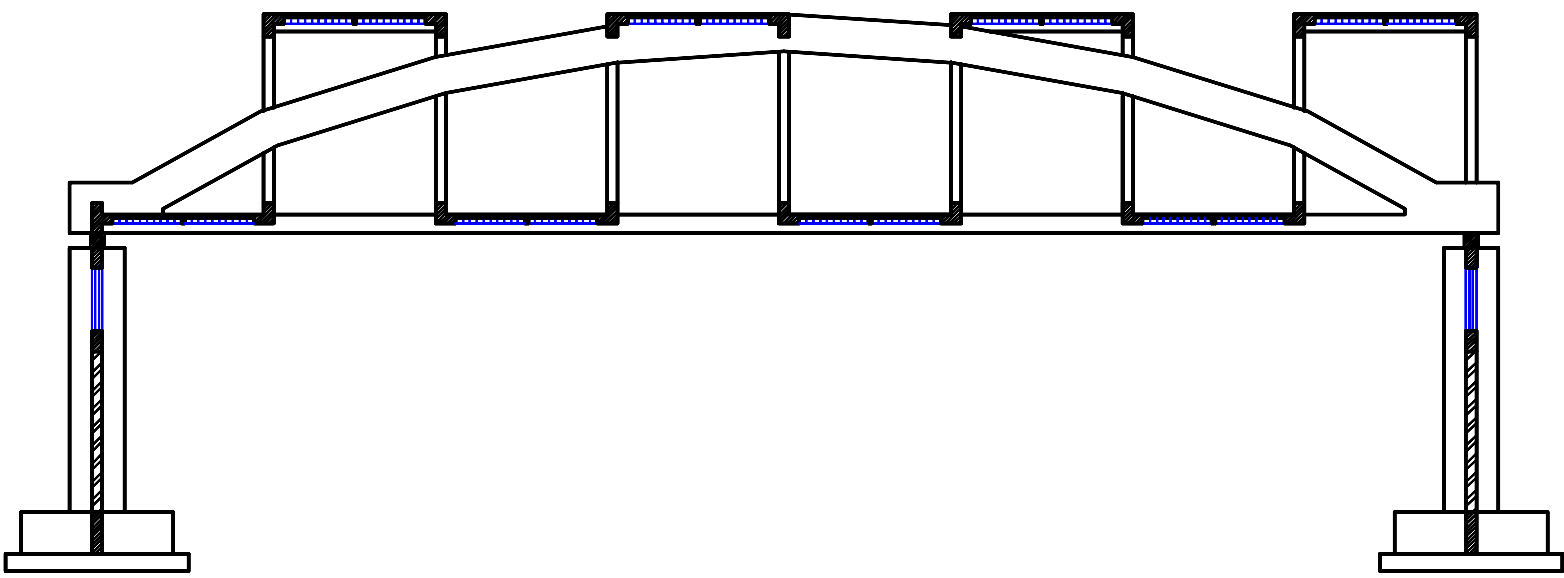

S
L


ELEVATION



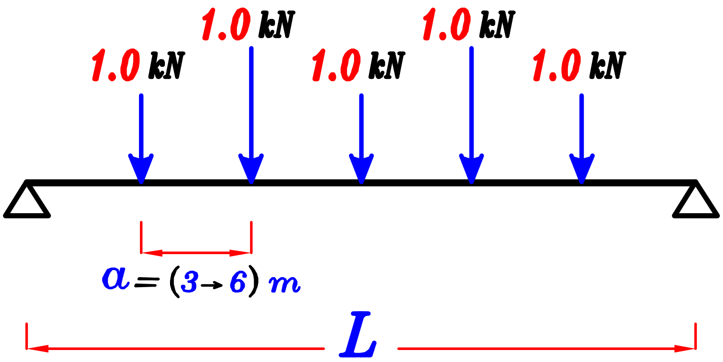




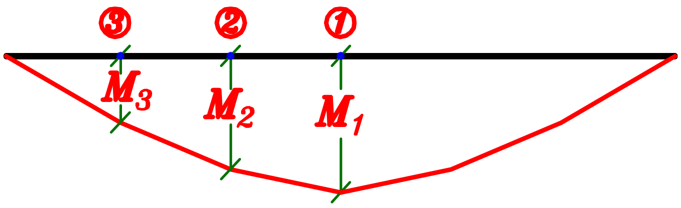


Drawing Arch Girder.

نرسم شكل الـ Arch بحيث يكون شكله مقلوب الـ B.M.D.
لكى يكون الـ B.M.D. ≃ Zero تقريباً.
فتكون كل القوى المؤثره عليه (N.F.) فقط.



– نرسم كمره بنفس طول الـ Arch
و نضع عليها Concentrated Loads
قيمتها 1.0 kN فى أماكن الكمرات



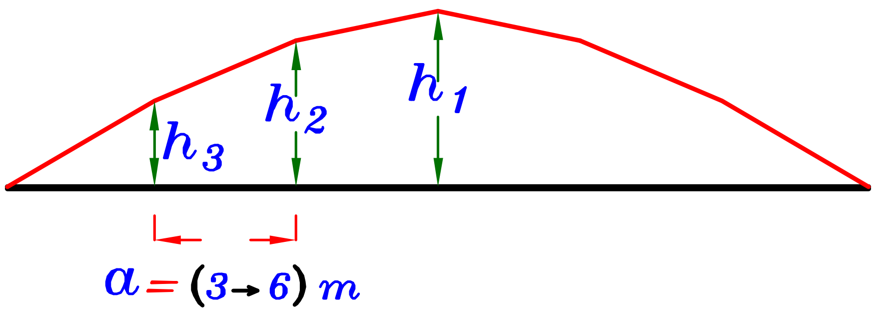
– ثم نرسم الـ B.M.D. Get M_1 , M_2 , M_3

– نختار أكبر أرتفاع للـ Arch $h_1 \simeq \dfrac{L}{6 \rightarrow 8}$

For Point ① $\longrightarrow$ $M = M_1$, $h = h_1$

For Point ② $\longrightarrow$ $M = M_2$, $h = h_2 = \dfrac{M_2}{M_1} * h_1$

For Point ③ $\longrightarrow$ $M = M_3$, $h = h_3 = \dfrac{M_3}{M_1} * h_1$



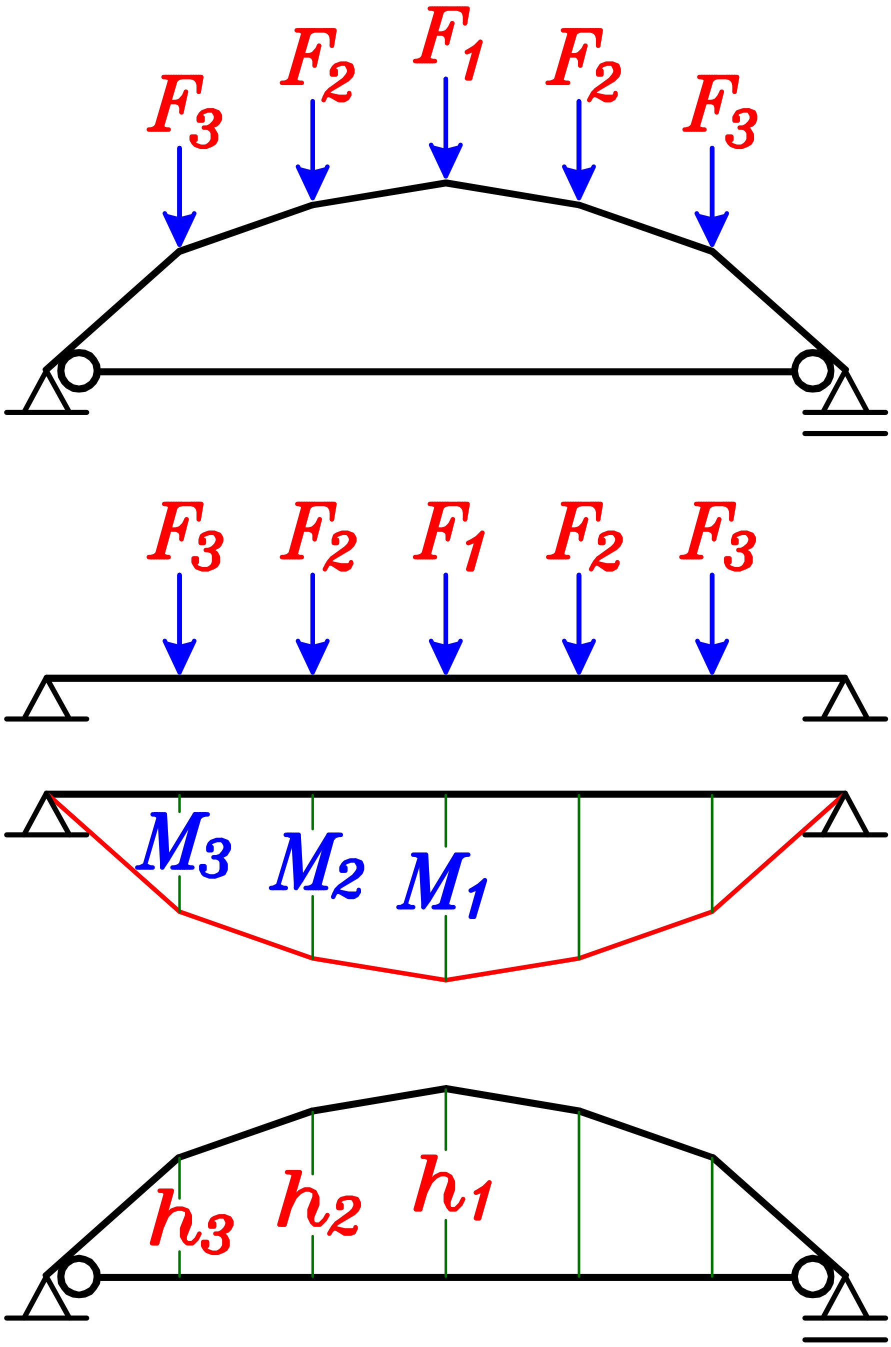
Special Case.

فى بعض الاحيان ممكن ان تكون الاحمال على ال **Joints** غير متساويه .

و فى هذه الحاله لكى نحدد ارتفاعات ال **Arch Girder**

يجب اولا ان نحدد قيمه الحمل على كل **Joint** ثم نحدد ال **moment** الفعلى عند كل **Joint**

و منها نحدد ارتفاعات ال **Arch Girder**

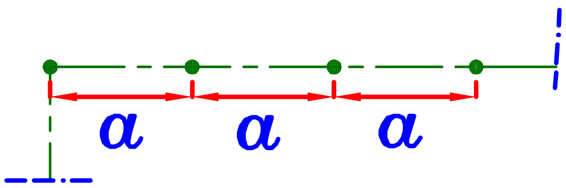


$$\boxed{M_1 : M_2 : M_3 = h_1 : h_2 : h_3}$$

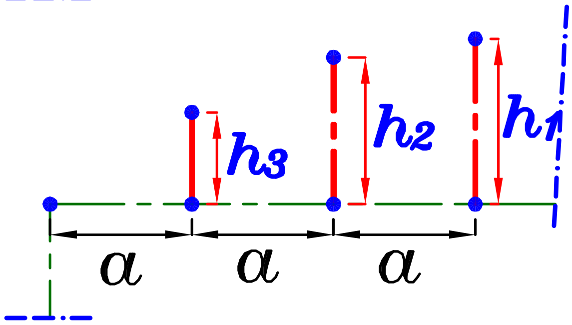
خطوات رسم ال Arch Girder

١ - نرسم C.L. للـ Tie و العمود

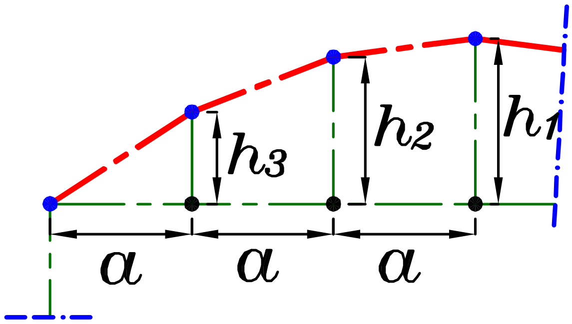
٢ - نحدد على C.L. ال Tie
مكان ال joints كل مسافه a



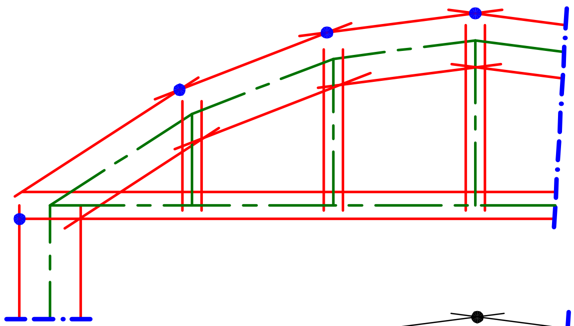
٣ - نرسم h عند كل joint



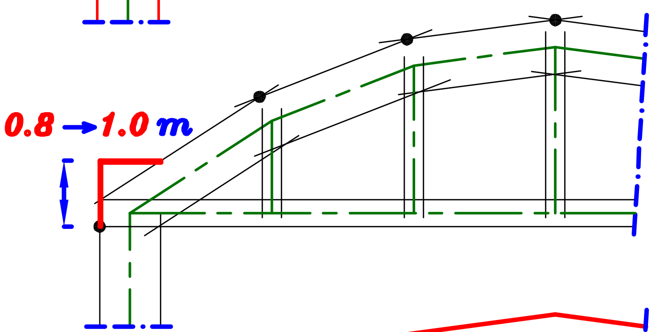
٤ - نوصل C.L. للـ Girder



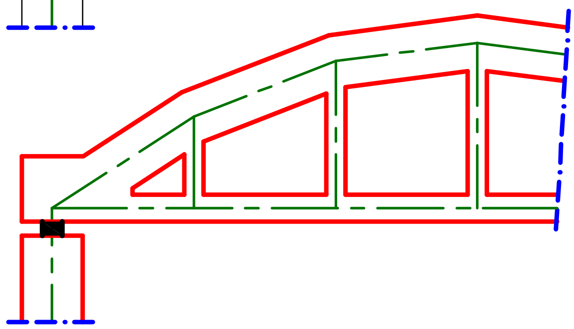
٥ - نرسم تخانه ال members
بخط خفيف و تحديد نقط التقاطع



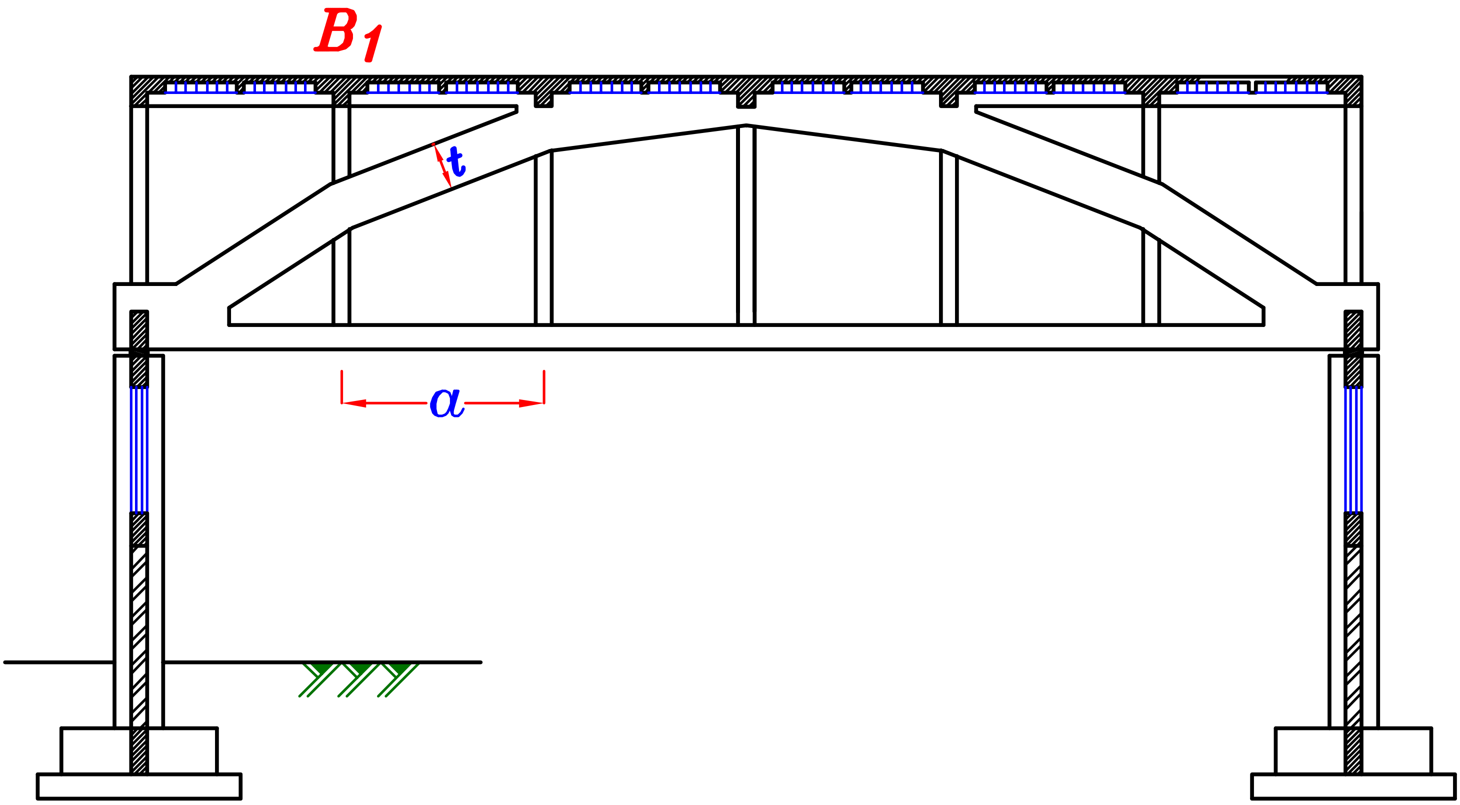
٦ - نرسم خط رأسى من نقطه تقاطع
العمود مع ال Tie بارتفاع من
($0.8 \rightarrow 1.0$ m) لمقاومه ال Shear
ثم نرسم خط أفقى

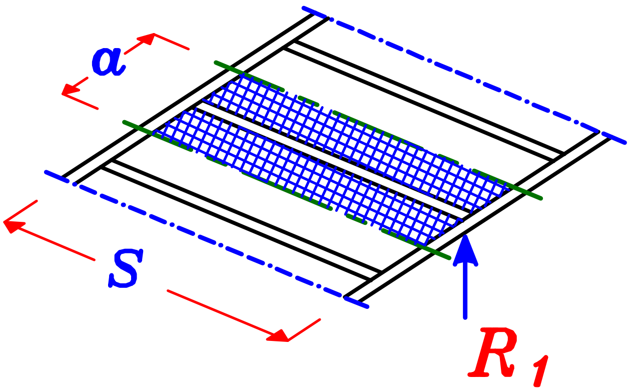


٧ - نرسم خط تقيل عند حدود ال
Arch Girder



Loads on Arch Girder.





① Get Loads on Beam B_1

$$w = O.W._{(beam)} + w_s * a$$

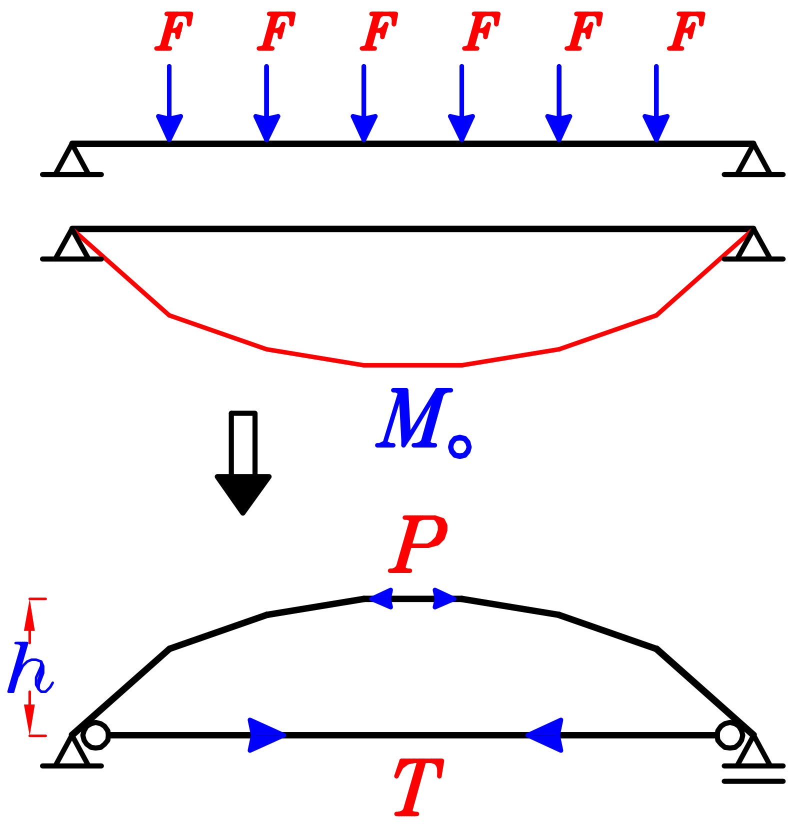
$$w = O.W._{(beam)} + \left(\frac{w_{rib}}{s}\right) * a$$

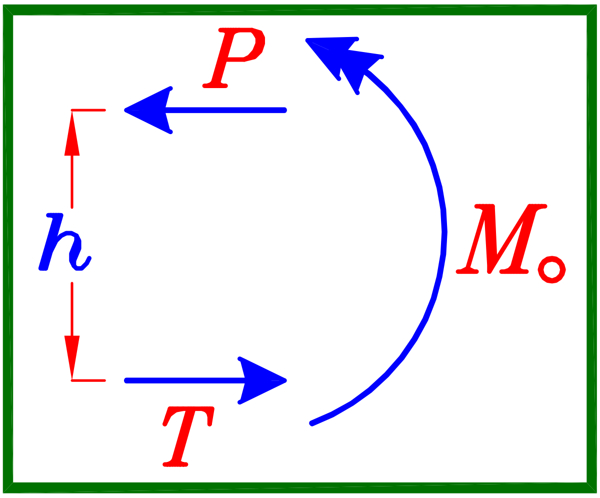
$$R = w * S$$

② $o.w._{(Arch+Tie+Post+Hanger+Top\ beam)} \simeq 17.5$ kN\m` (U.L.)

③ $F = o.w. * a + R$

Concept of Arch Girder.





$$\boxed{P = T = \frac{M_o}{h}}$$

تعتمد فكره الـ *Arch Girder* على تحويل الـ *Bending moment* الى *Couple*

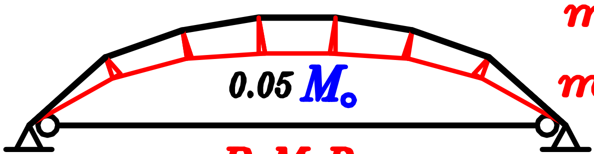
اى الى *Tension Normal Forces* & *Compression Normal Forces*

و ذلك للتوفير لانه عند تصميم قطاع عليه *pure Compression* ستكون كميه الخرسانه

و الحديد قليله مما يعمل على تقليل ثمن الـ *member*

و عند تصميم قطاع عليه *pure Tension* تكون كميه الحديد كبيره و كميه الخرسانه

قليله و تكون ايضا نسبيا ثمن الـ *member* أقل .

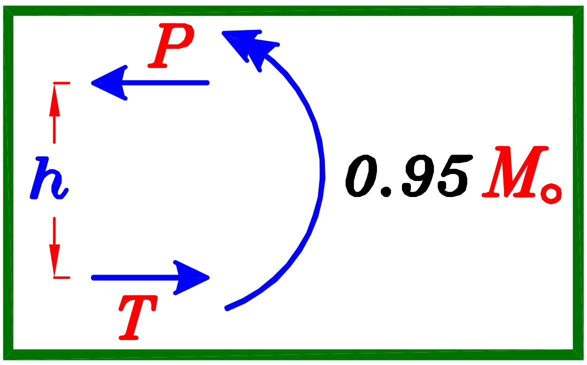


B.M.D.

اذا حدثت استطاله بسيطه لـ *Tie* سيحدث *moment*

بسيط قيمته فى حدود $0.05\,M_o$ اذا قيمه الـ *moment*

الذى سيتحول لـ *couple* يساوى تقريبا $0.95\,M_o$



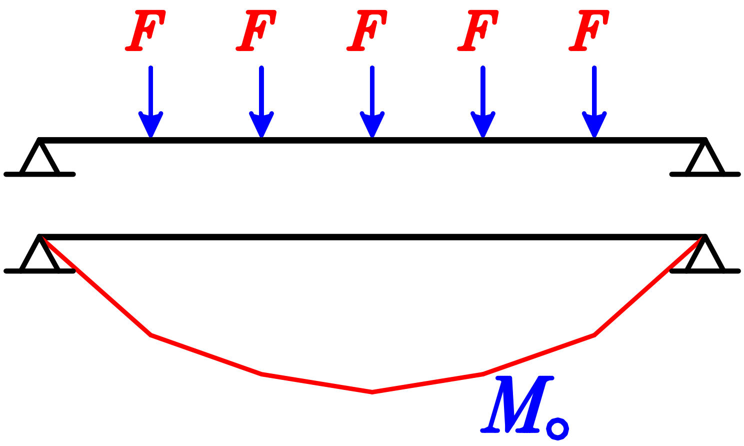
$$\boxed{P = 0.95\ \frac{M_o}{h}}$$

$$\boxed{T = 0.95\ \frac{M_o}{h}}$$

Solving Arch Girder.

نفرض وجود كمره تخيليه أفقيه .
لها نفس ال **span** الافقى للـ **Frame**

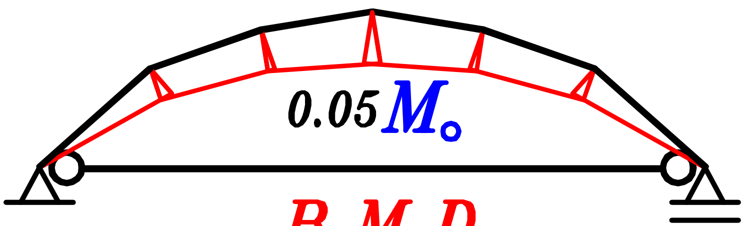
نحسب قيمه أكبر **moment**
للكمره التخليه و يسمى ((M_o))



$$\boxed{M = 0.05 \ M_o}$$

(**From Extension of the Tie**)

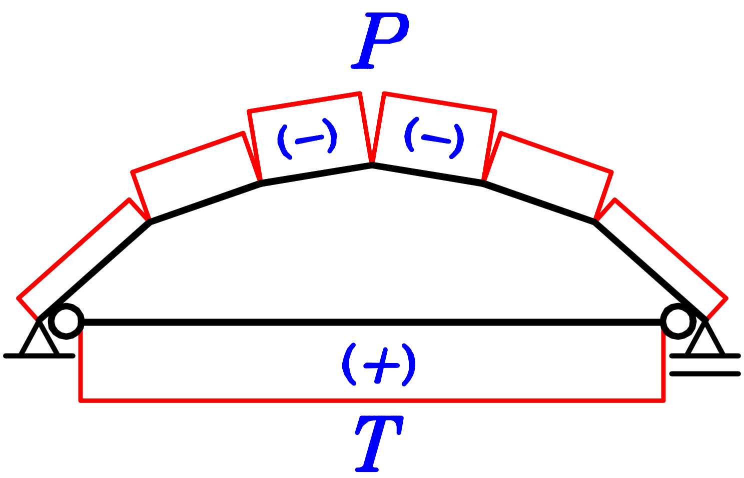
نتيجه لحدوث استطاله بسيطه فى ال **Tie**
يحدث عزم على ال **Frame** قيمته فى حدود $0.05 \ M_o$



B.M.D.

$$\boxed{P = 0.95 \ \frac{M_o}{h}}$$

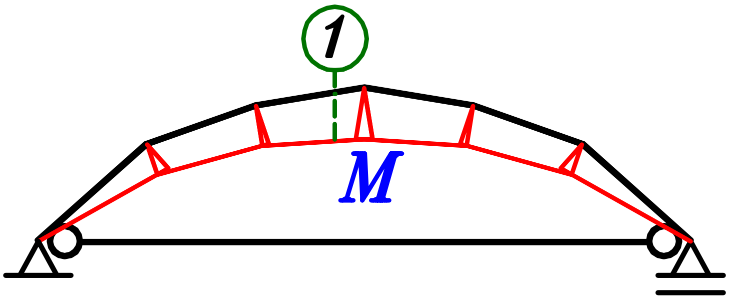
$$\boxed{T = 0.95 \ \frac{M_o}{h}}$$



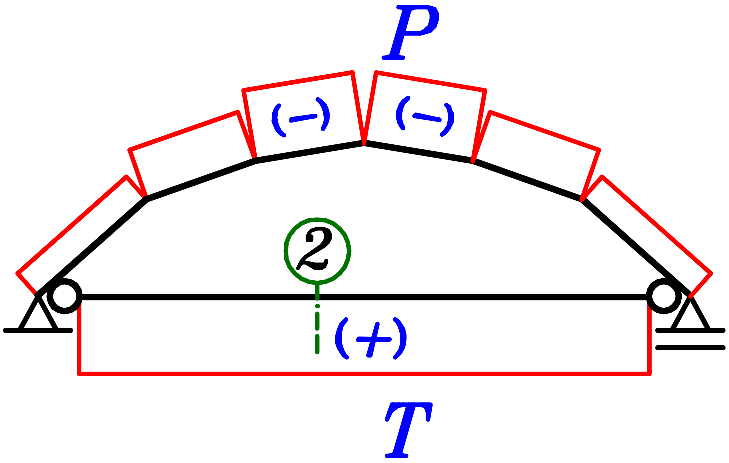
N.F.D.

Design sections of Arch Girder.

B.M.D.



N.F.D.



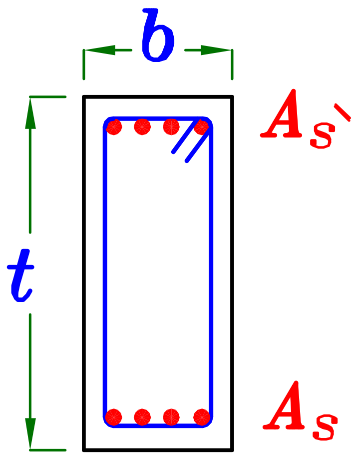
Sec. 1   M , P

$$P = 0.95 \, \frac{M_o}{h}$$

$$M = 0.05 \, M_o$$

(From Extension of the Tie)

(Use I.D.) $A_s = A_s'$

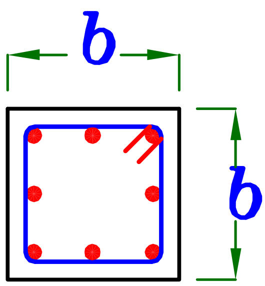


Sec. 2   Tie

$$T = 0.95 \, \frac{M_o}{h}$$

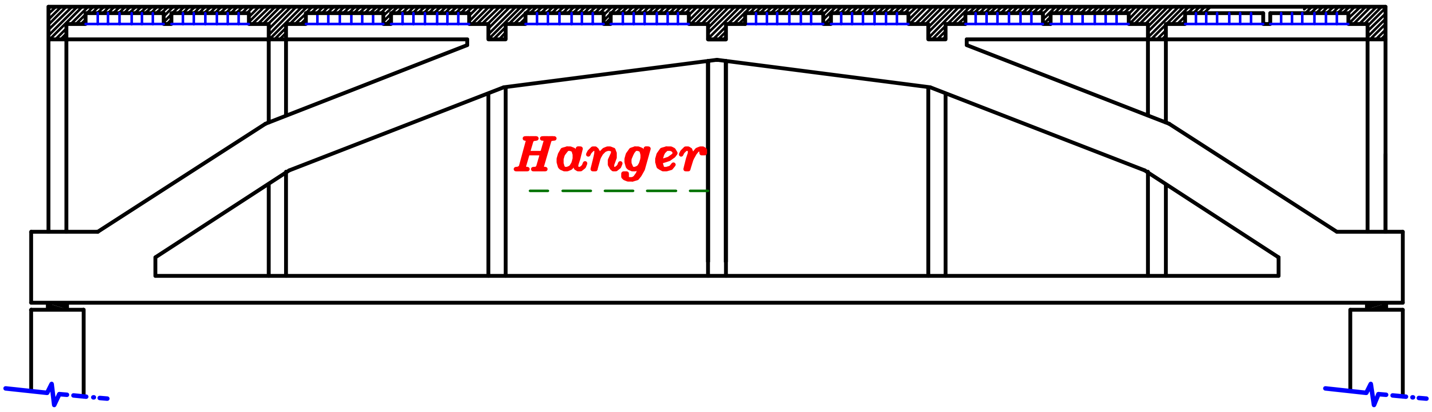
$$A_s = \frac{T}{F_y \backslash \gamma_s} = (\text{Total area of steel})$$

$$A_c = (b * b)$$



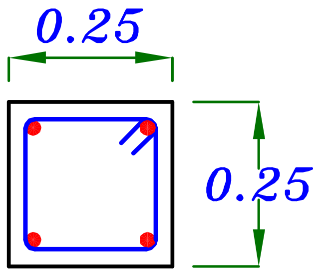
Sec. ③ Hanger

١ - إذا كانت البلاطه علويه.

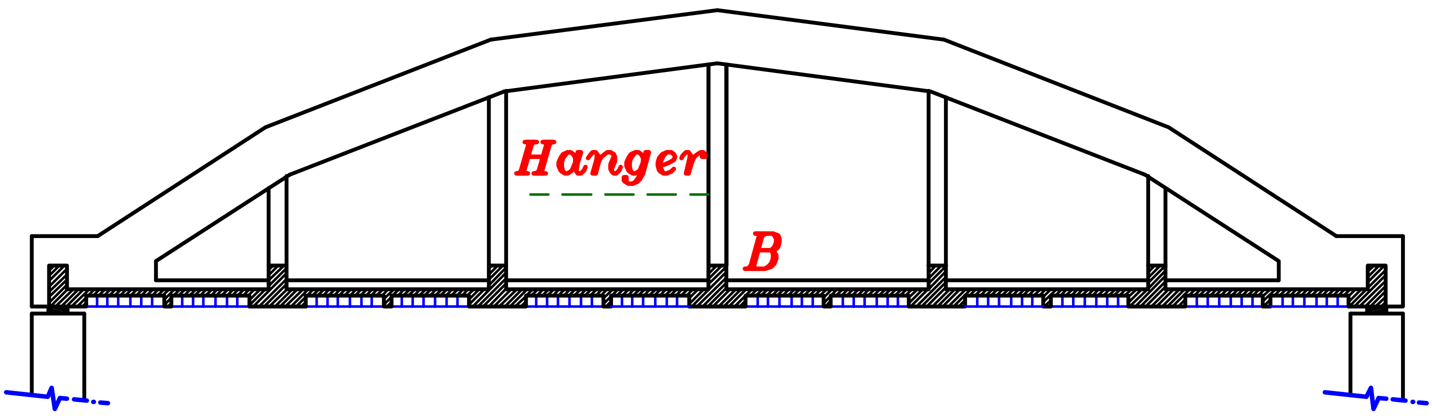


$$T = O.W._{(hanger)} \simeq 3.50 \ kN$$

$$\boxed{A_s = 4 \phi 12}$$

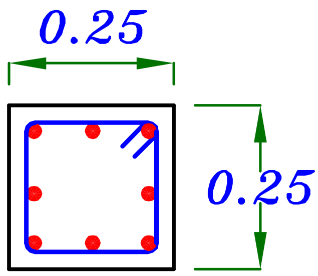


٢ - إذا كانت البلاطه سفليه.



$$T = O.W._{(hanger)} + \text{Reaction of beam } B$$

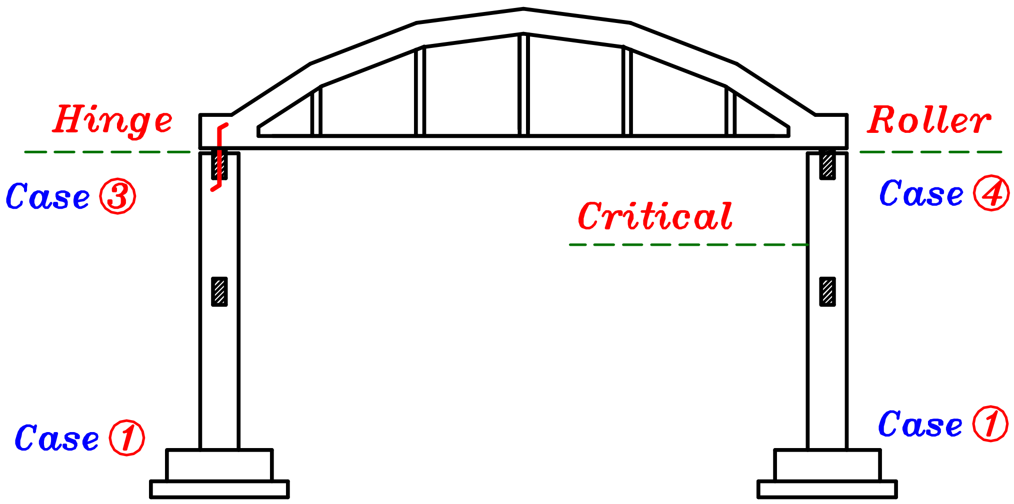
$$\boxed{A_s = 8 \phi 12}$$



Sec. ④ Column.

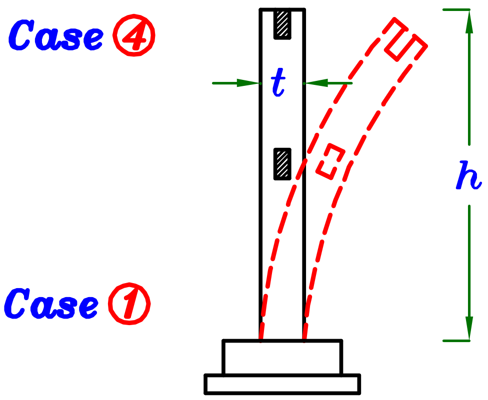
Design the critical Col. at the Roller support.

$$\boxed{P = \frac{\sum F}{2}}$$



Check Buckling.

① In plane.

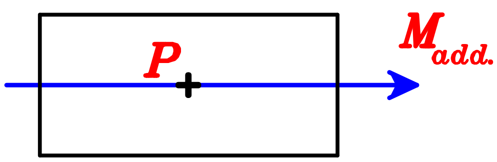


$H_{\circ} = h$

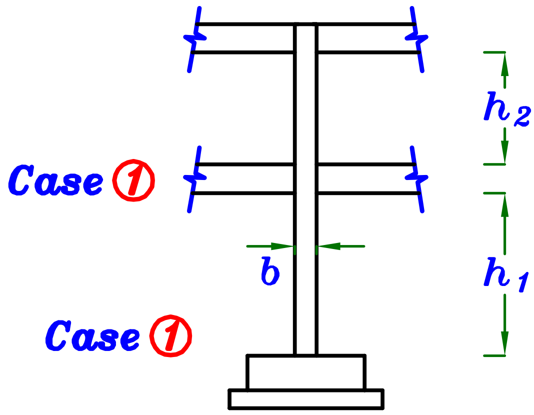
$$\lambda_b = \frac{2.2 * H_{\circ}}{t}$$

IF $\lambda_b \leqslant 10 \xrightarrow{\text{Designed}} P$ only

$\lambda_b > 10 \xrightarrow{\text{Designed}} P\,,\ M_{add.}$



② Out of plane.

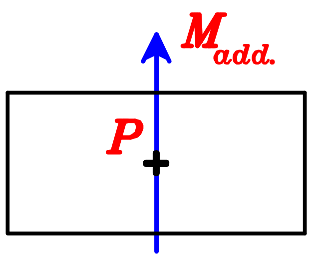


$H_{\circ} =$ The bigger of h_1 , h_2

$$\lambda_b = \frac{1.2 * H_{\circ}}{b}$$

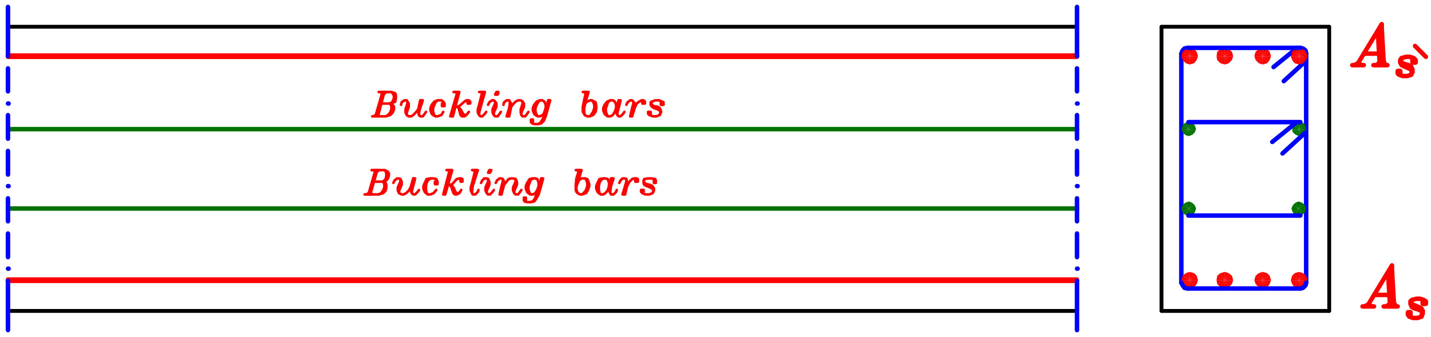
IF $\lambda_b \leqslant 10 \xrightarrow{\text{Designed}} P$ only

$\lambda_b > 10 \xrightarrow{\text{Designed}} P\,,\ M_{add.}$

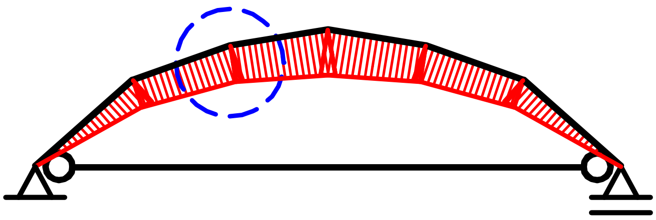


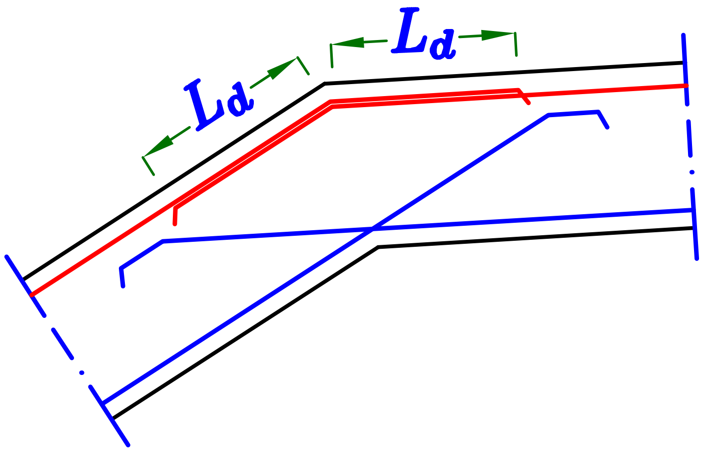
Reinforcement Notes.

1- For Member subjected to Moment & Compression.



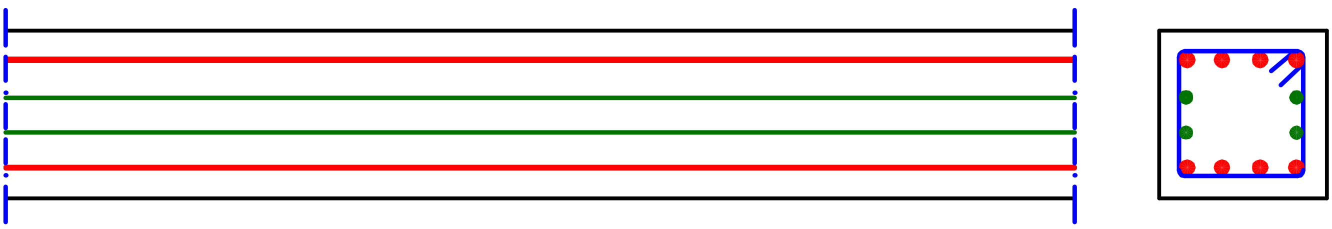
و اذا زاد طول السيخ عن ١٢,٠ م نعمل وصله تراكب **Lap splice**
و الاضمن مد الحديد مسافه L_d من بعد ال C.L. من الجهتين .





2- For Member subjected to pure Tension. (Tie)

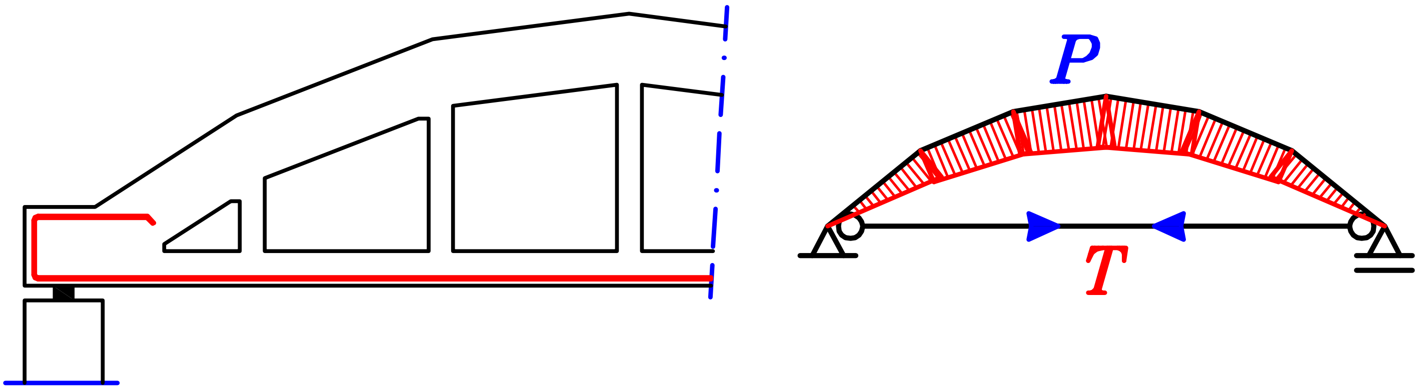
و اذا زاد طول السيخ عن ١٢,٠ م المفروض عمل وصله ميكانيكيه او وصلات لحام
و لصعوبه رسم تفاصيل ذلك فى هذا الملف فسنضطر ان نرسم تسليح ال **Tie**
عباره عن اسياخ طويله طولها بنفس طول ال **Tie** حتى اذا زاد طولها عن ١٢,٠ م



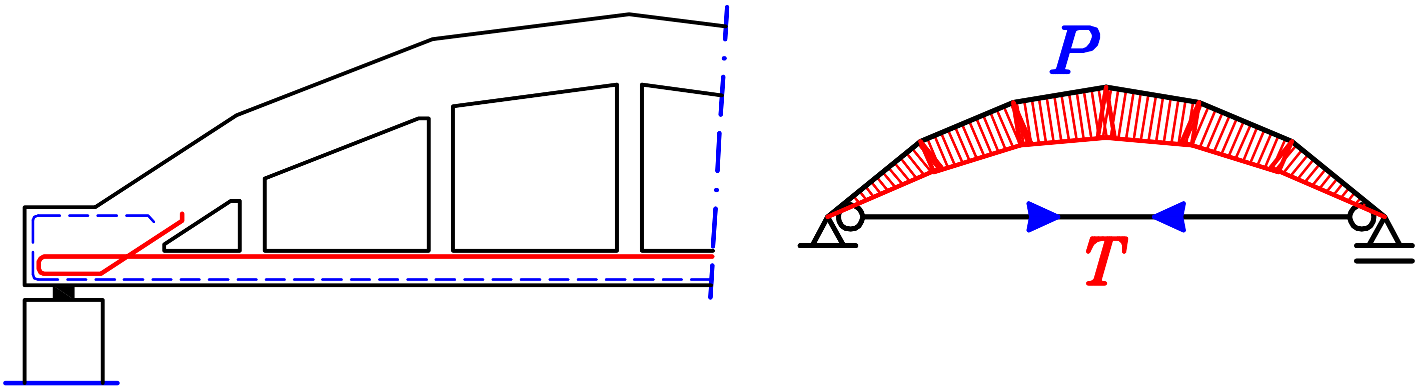
لعمل وصلات فى ال (Tie) أنظر فى هذا الملف صفحه (90)

Reinforcement of Arch Girder.

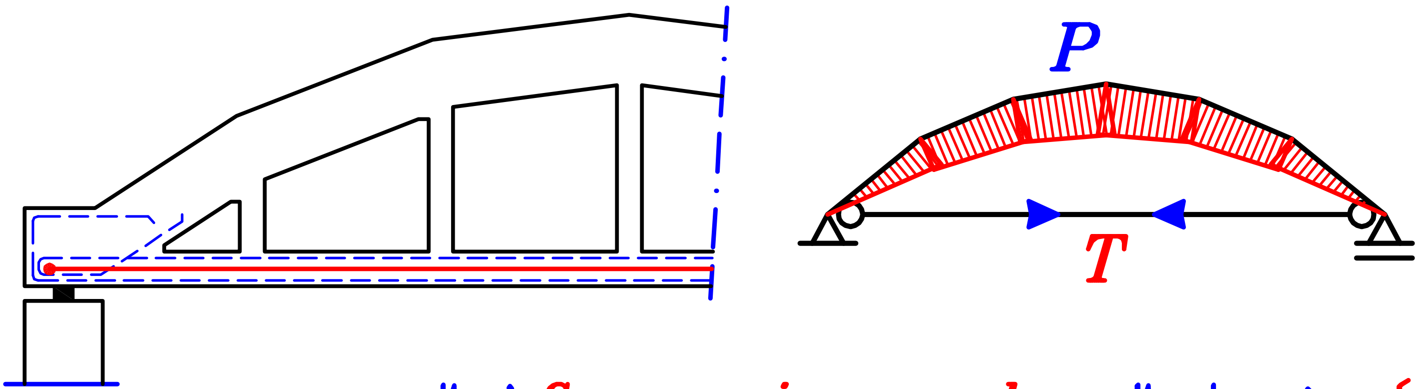
١ ـ بعد رسم الخرسانه نضع الحديد السفلى لل *Tie* مع مراعاه تكملته فى الدراسه من الاول للاخر



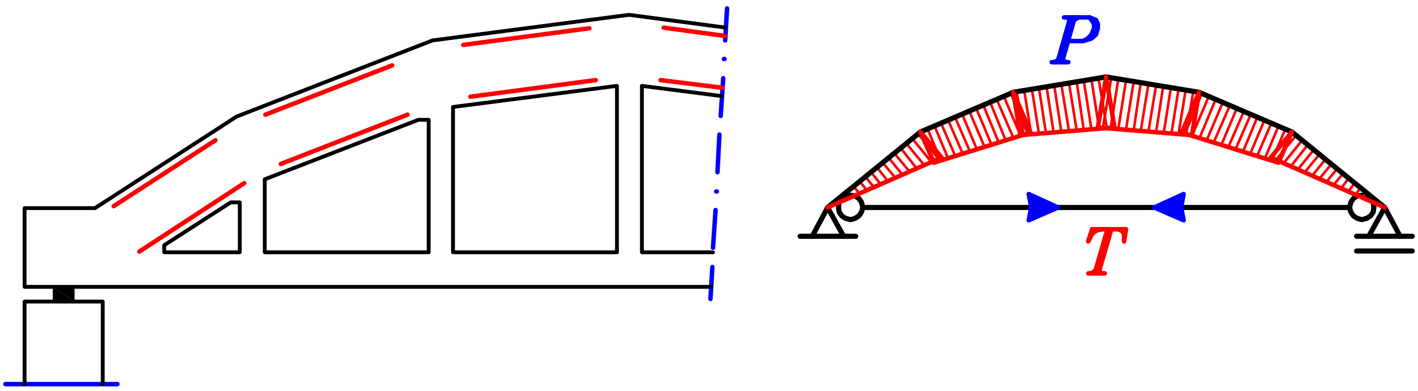
٢ ـ نرسم التسليح العلوى لل *Tie* مع مراعاه تكملته فى الدراسه من الاول للاخر



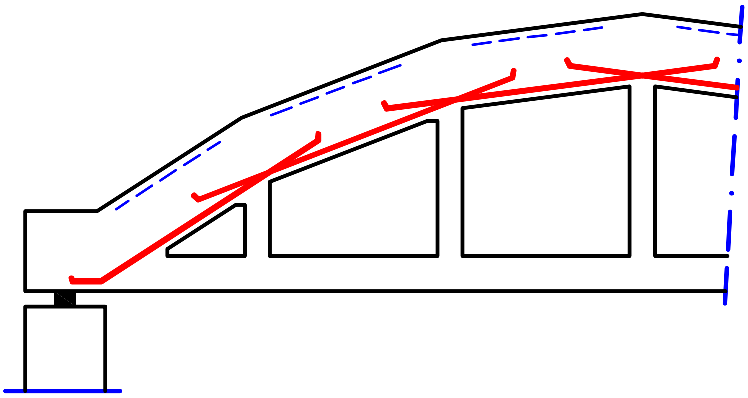
٣ ـ نرسم التسليح الاوسط لل *Tie* مع مراعاه تكملته فى الدراسه من الاول للاخر

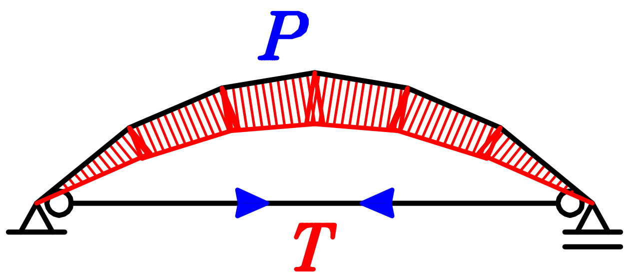


٤ ـ وضع تسليح ال *Compression members* فى الجهتين

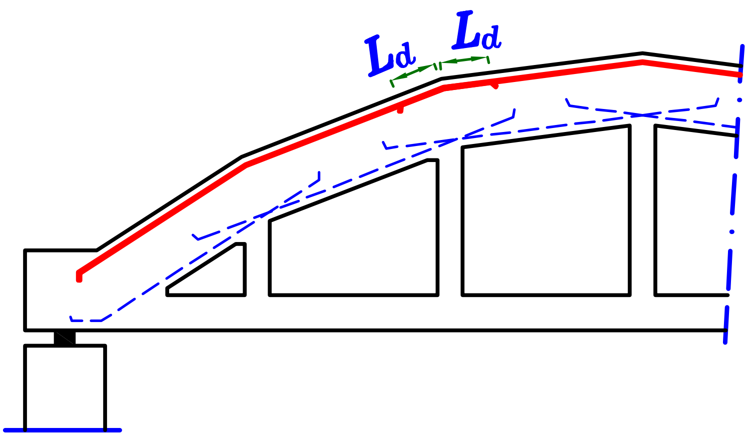


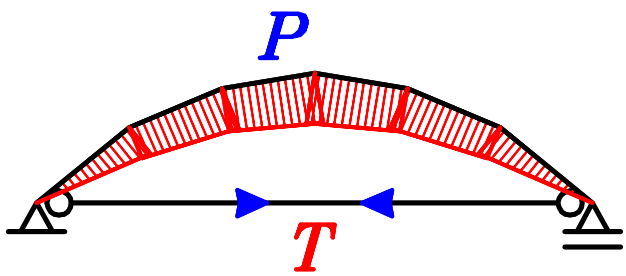
٥ – الحديد السفلى يمتد من الجهتين مسافه $L_d = 60\,\phi$ من الجهتين



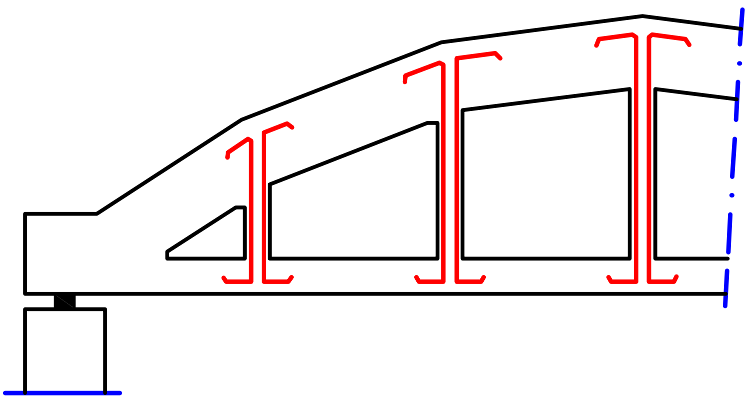


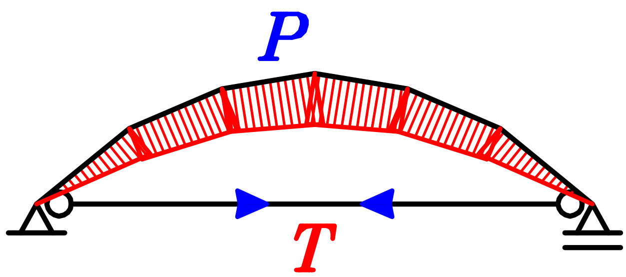
٦ – الحديد العلوى يمتد من الجهتين مسافه $L_d = 40\,\phi$ من الجهتين كل 2 joints



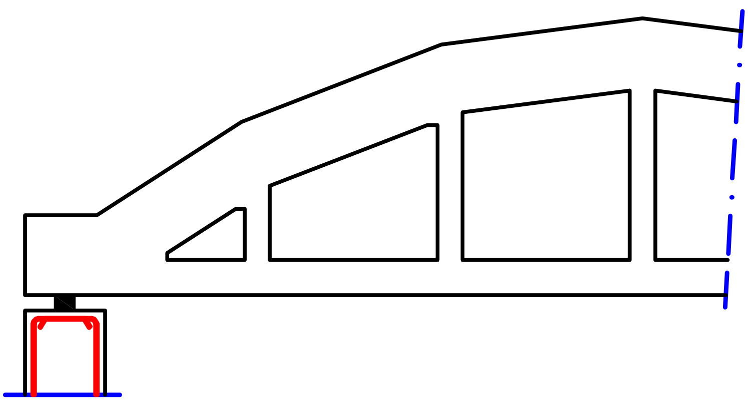


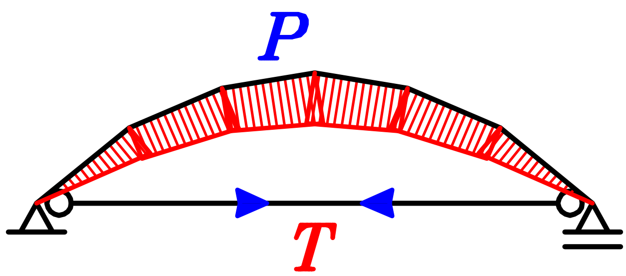
٧ – نضع تسليح الـ Hanger



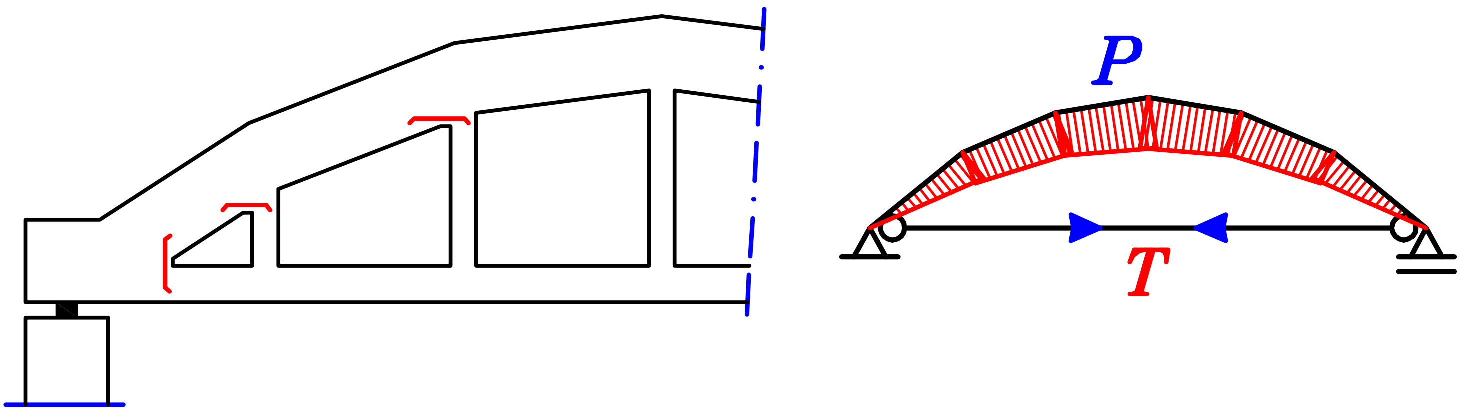


٨ – نضع تسليح العمود

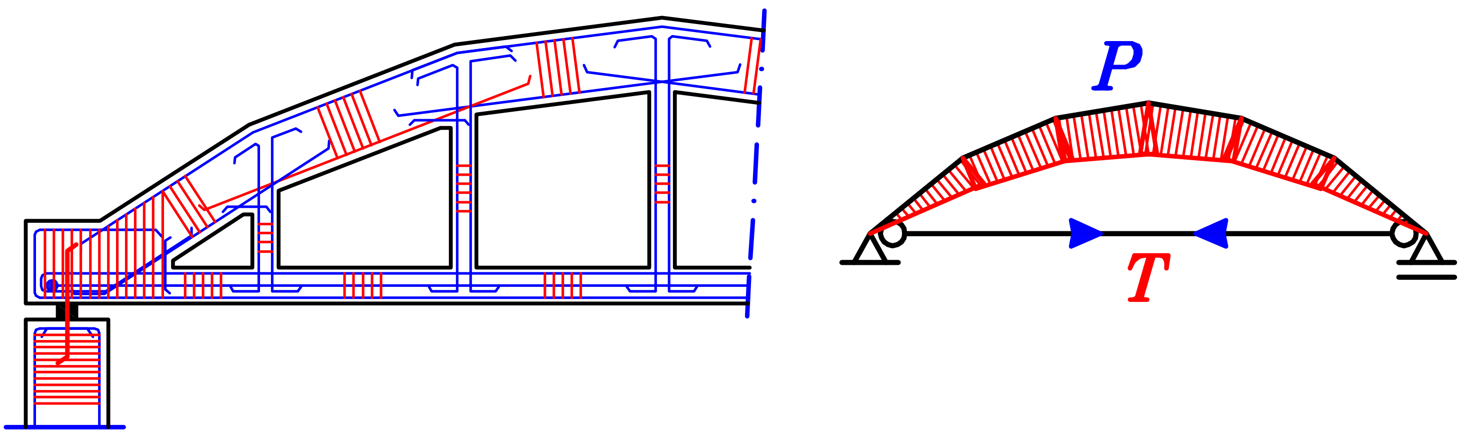


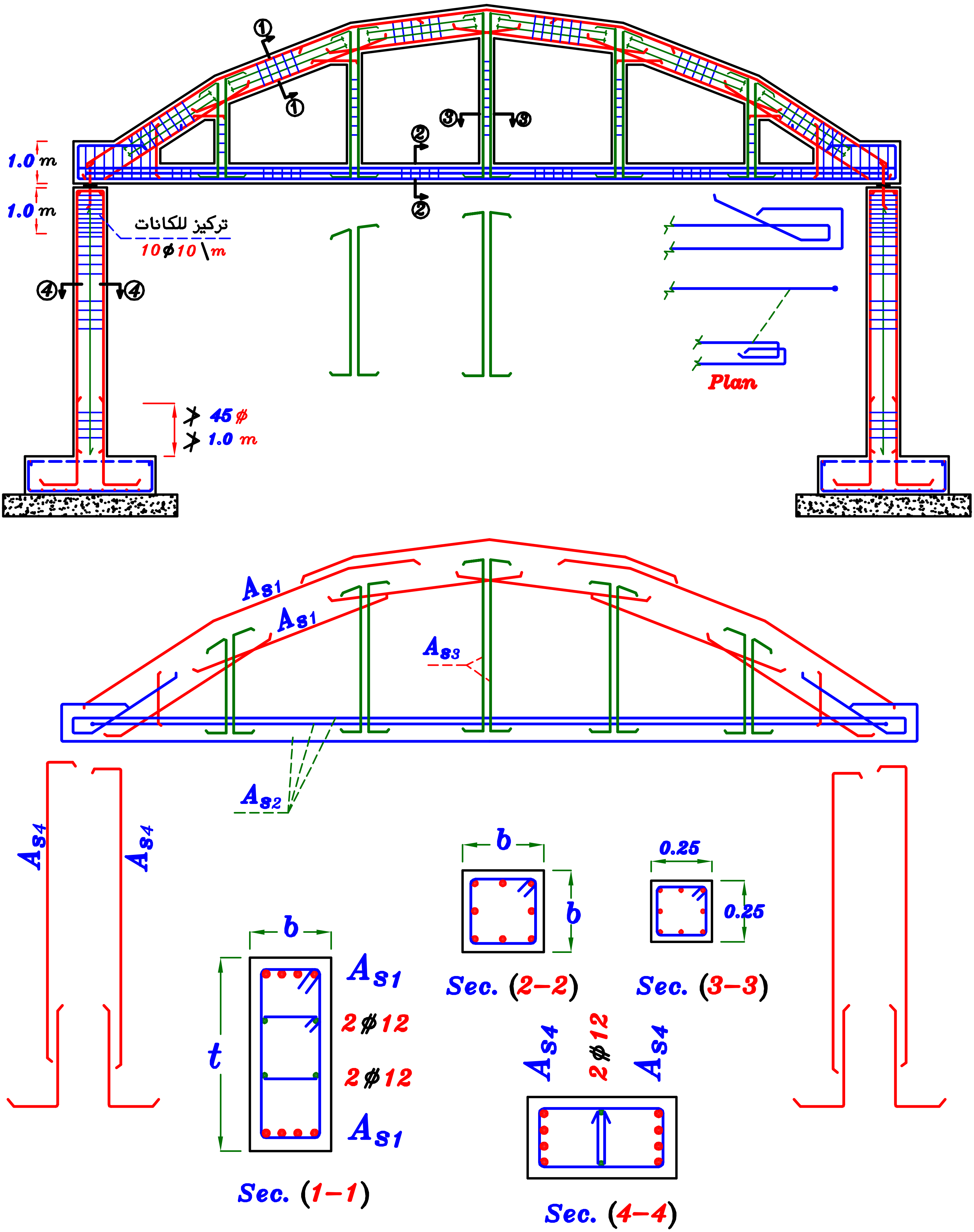


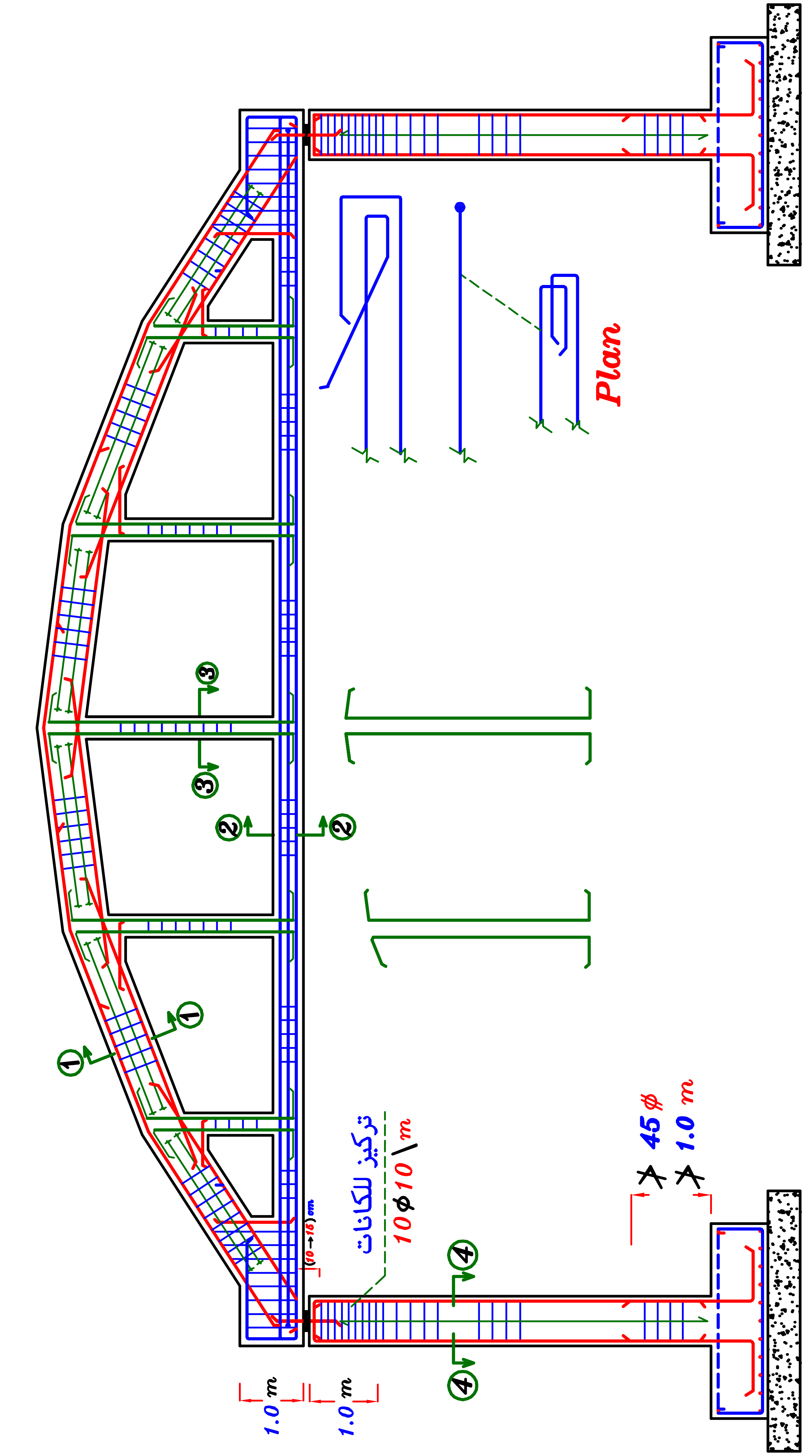
٩ – نضع تسليح بسيط فى حدود $2\phi 10$ عند الزوايا الحاده لمنع تشرخ الـ Cover



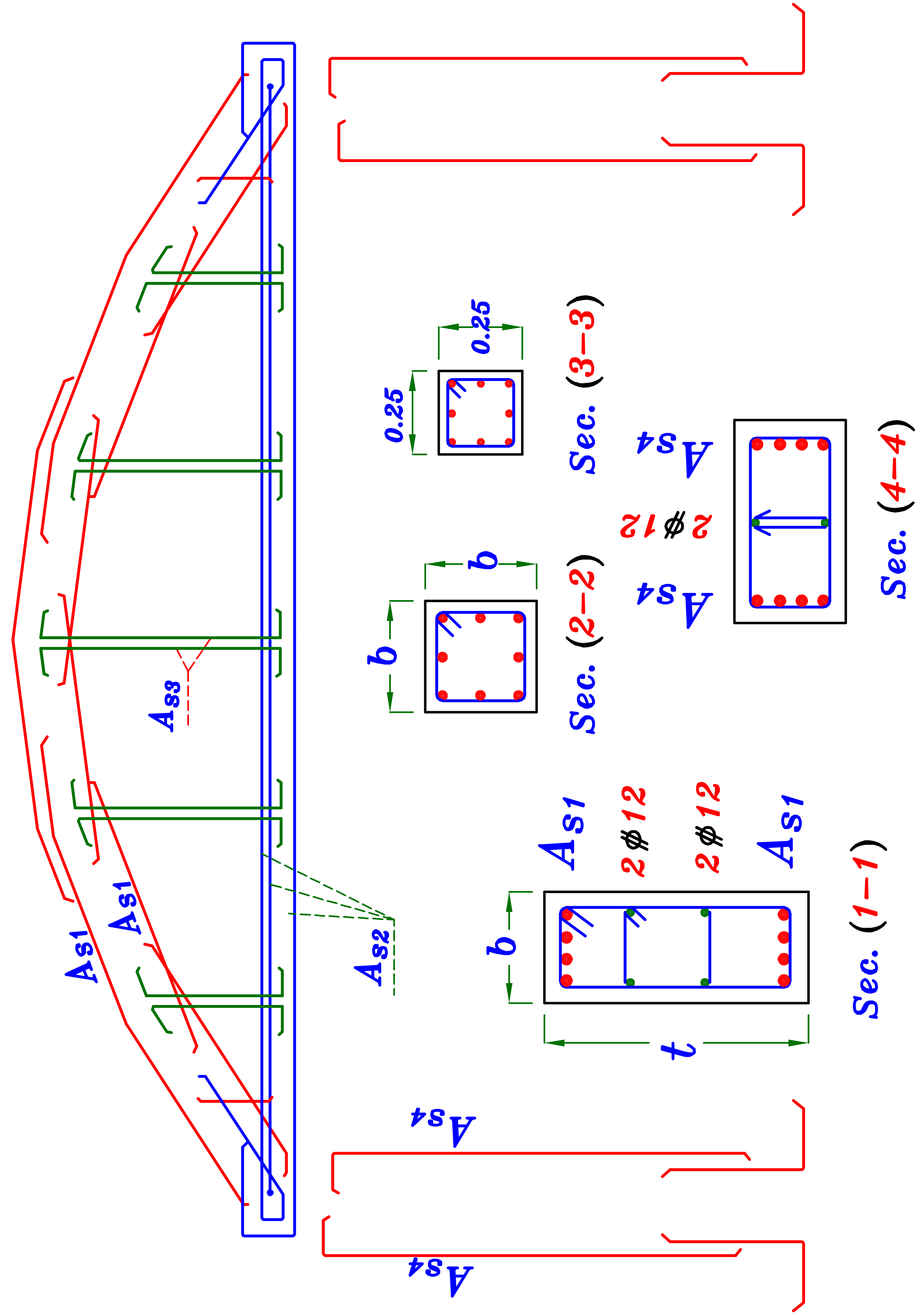
١٠ – يتم وضع الكانات مع تكثيف الكانات اعلى العمود لمقاومه Splitting Force




1.0 m
1.0 m
تركيز للكانات
10 ϕ 10 \ m
45 ϕ
1.0 m
Plan
As1
As1
As3
As2
As4
As4
b
t
As1
2 ϕ 12
2 ϕ 12
As1
Sec. (1–1)
b
b
Sec. (2–2)
0.25
0.25
Sec. (3–3)
As4
2 ϕ 12
As4
Sec. (4–4)



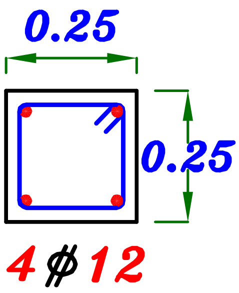
Plan
1.0 m
1.0 m
45 ϕ
1.0 m
تركيز للكانات
10ϕ10\m
(10→15) cm



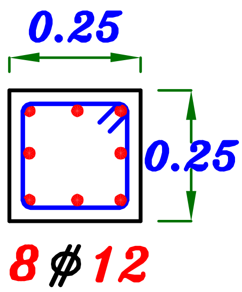
As1
As1
As2
As3
As4
As4
0.25
0.25
Sec. (3-3)
b
b
Sec. (2-2)
As4
2 ϕ 12
As4
Sec. (4-4)
As1
2 ϕ 12
2 ϕ 12
As1
b
t
Sec. (1-1)

في حاله أن البلاطه علويه

سنحتاج لوضع Posts لحمل الكمرات ال Secondary

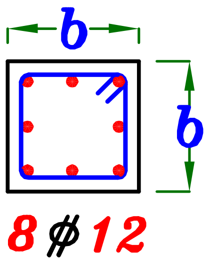


اذا كان ارتفاع ال Post $\leq 2.5\ m$ نأخذ قطاع ال Post



اذا كان $2.5\ m \leq$ ارتفاع ال Post $\leq 4.0\ m$

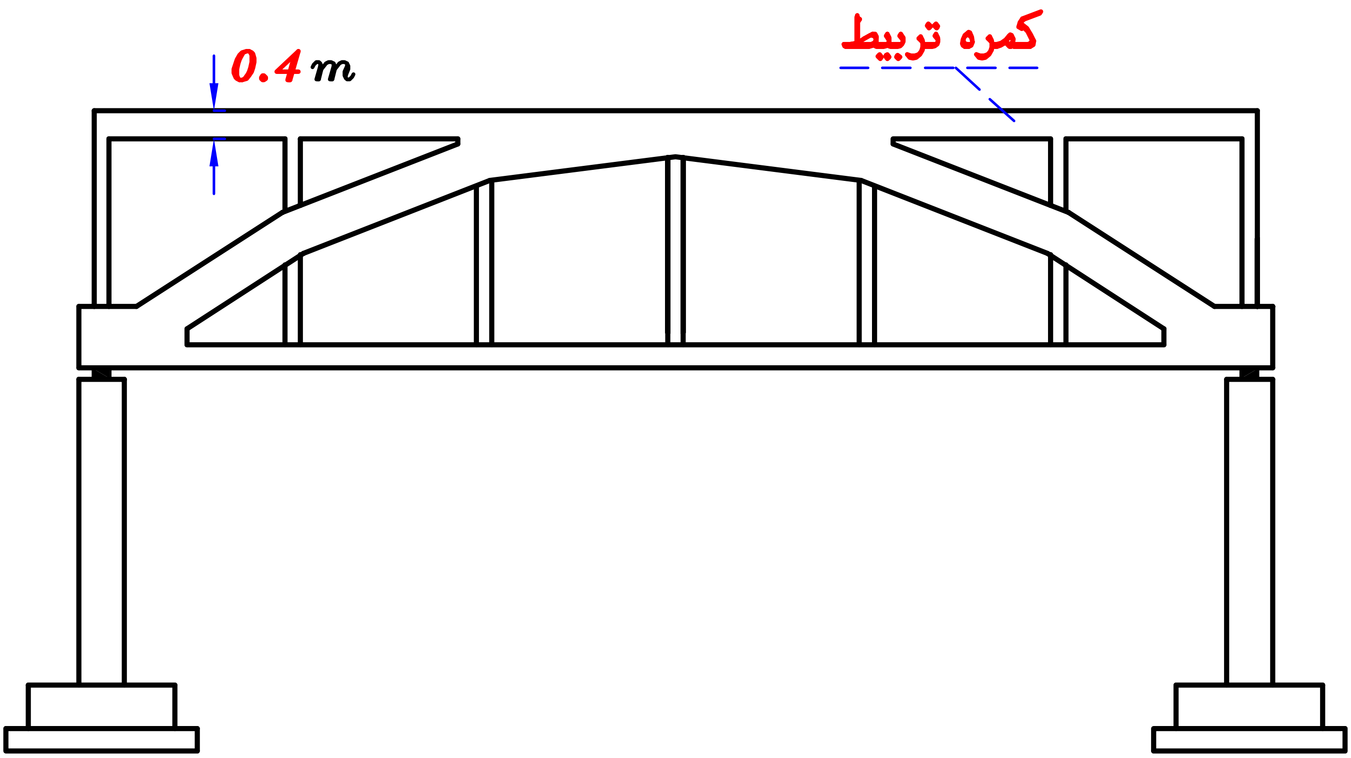
نأخذ قطاع ال Post



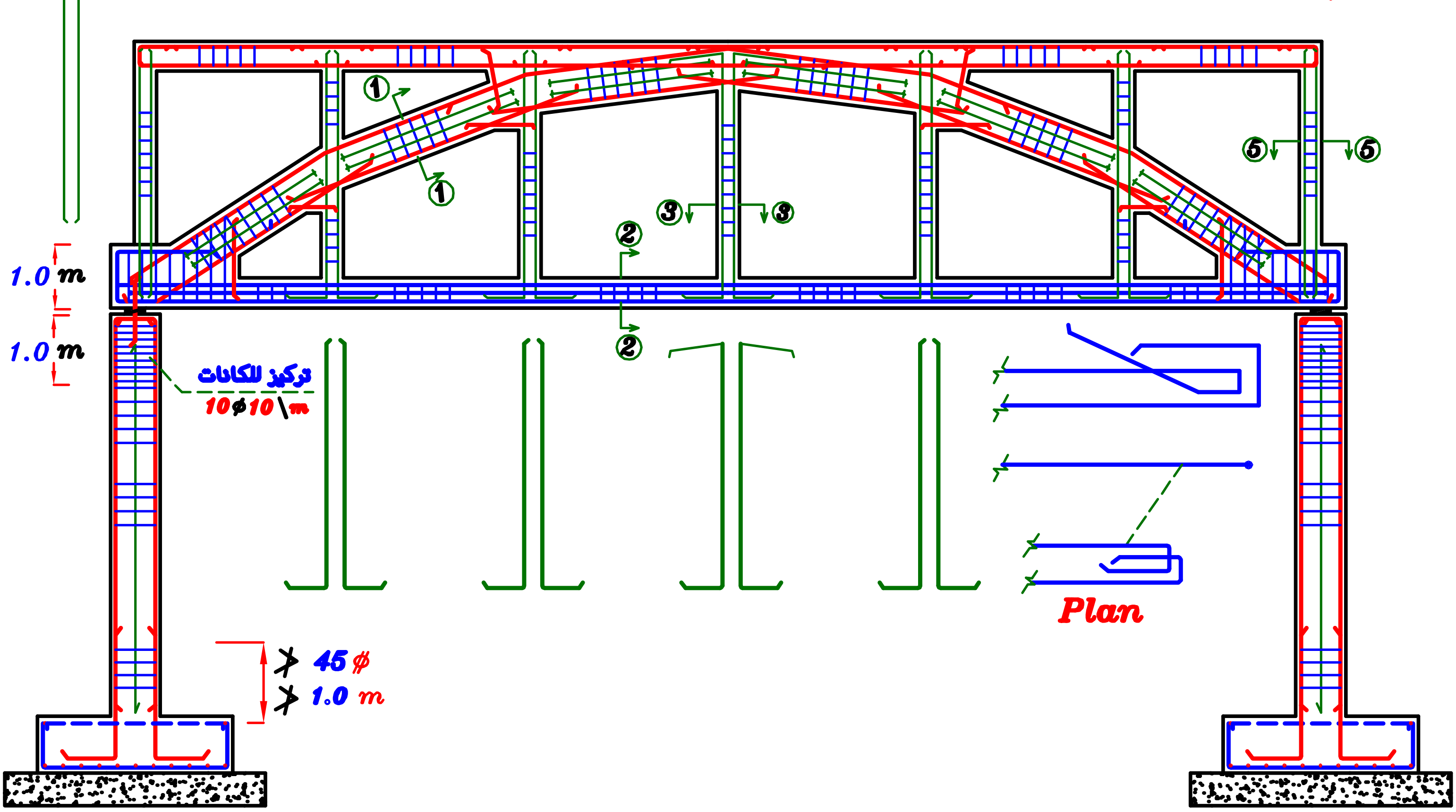
اذا كان ارتفاع ال Post $> 4.0\ m$ نأخذ قطاع ال Post

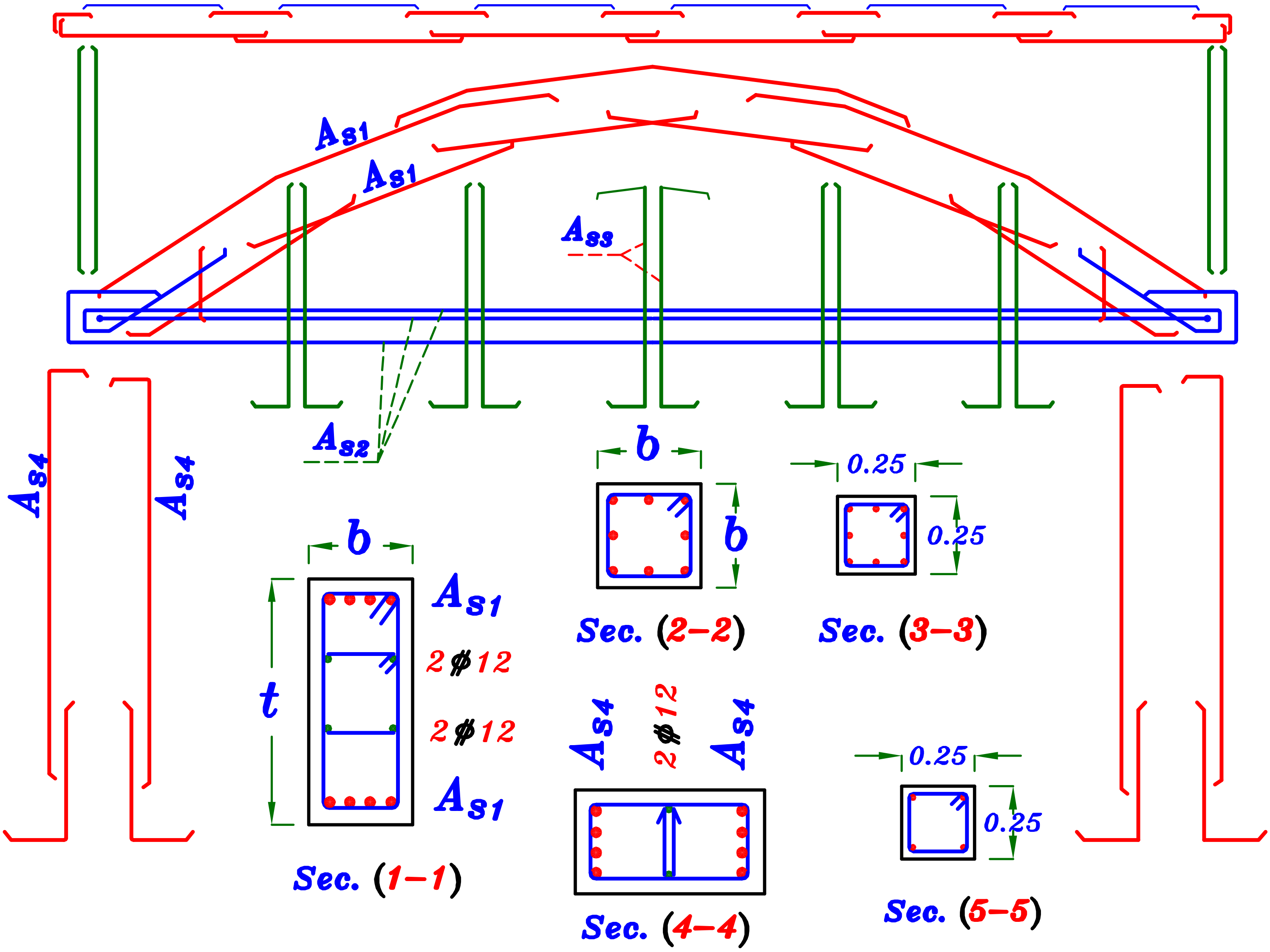
حيث b هى عرض ال Arc Girder

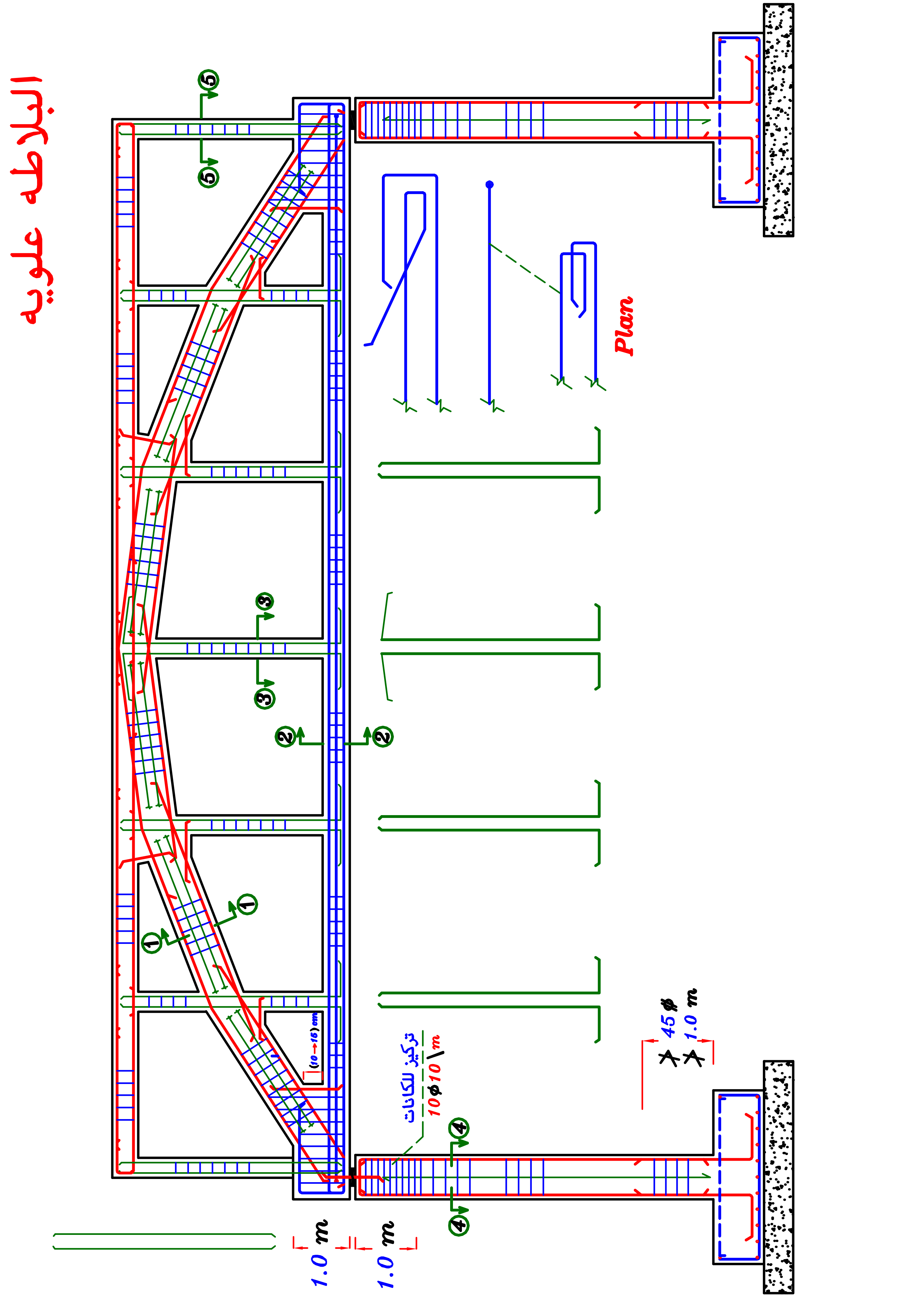
و نضع كمره افقيه لتربيط ال Posts عمقها 40 cm و تسليحها 2ϕ12 سفلى و علوى



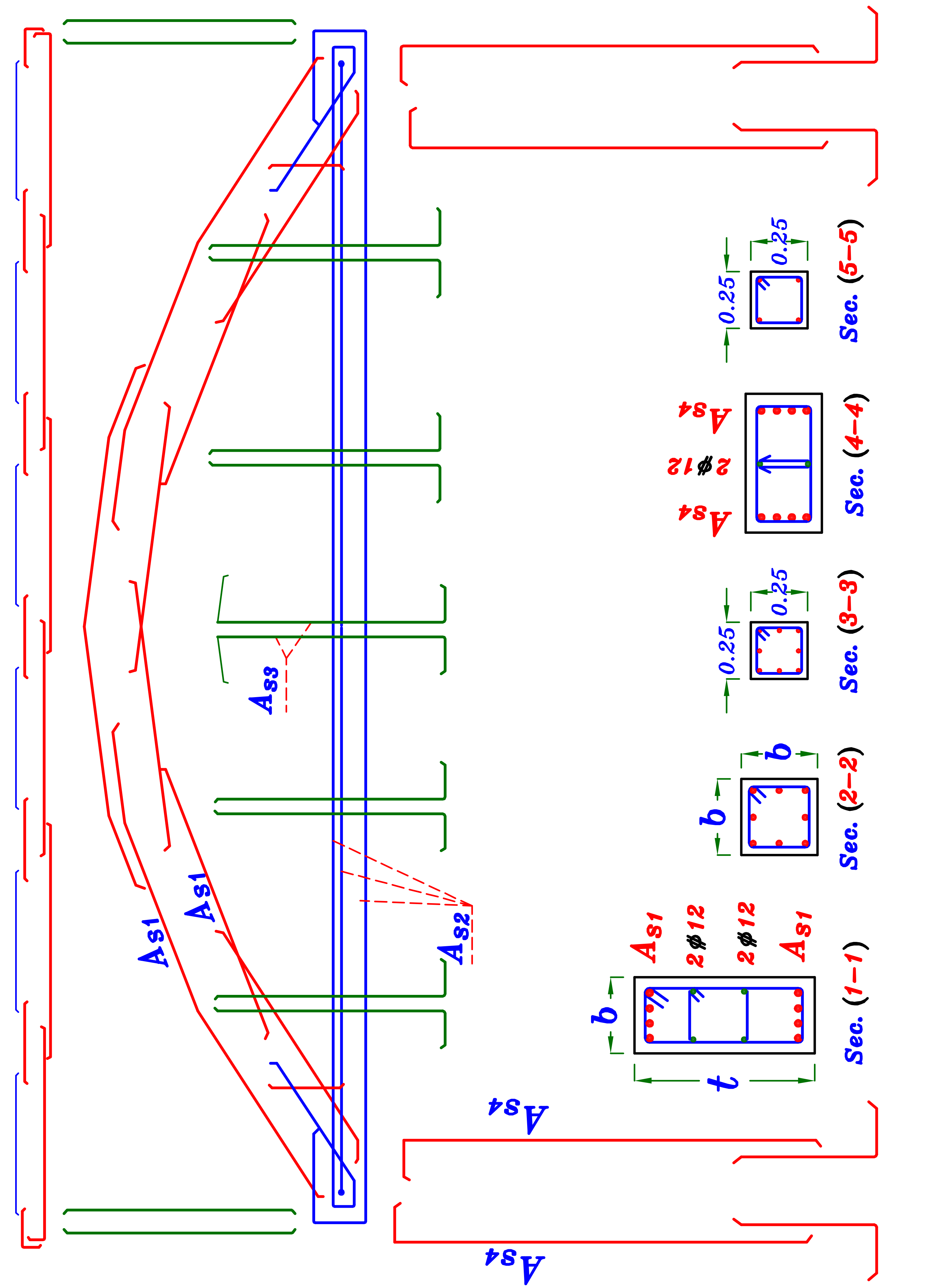
البلاطه علويه



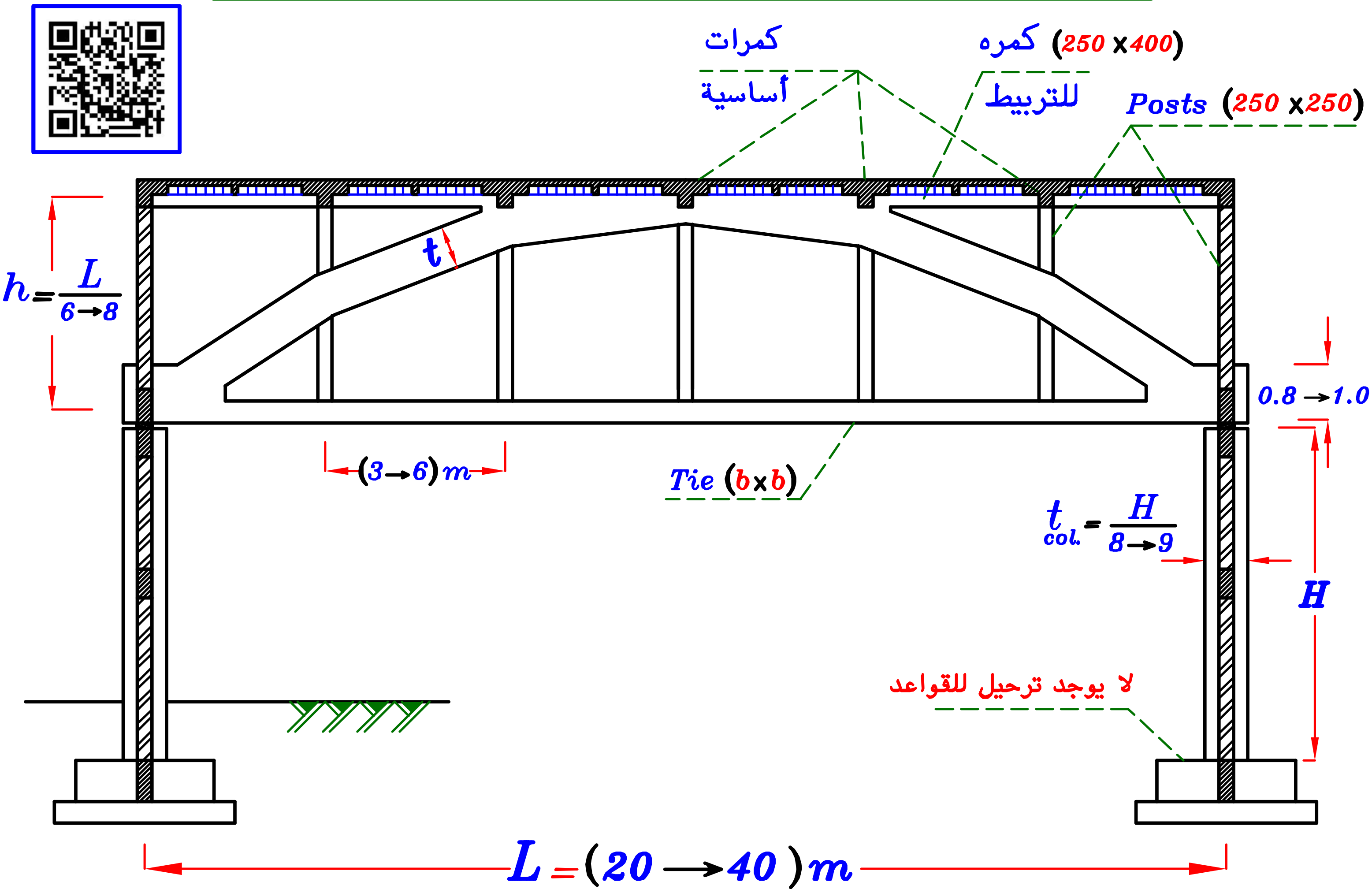




البلاطه علويه
Plan
تركيز للكانات
10φ10\m
45 φ
1.0 m
1.0 m
1.0 m
(10→15) cm

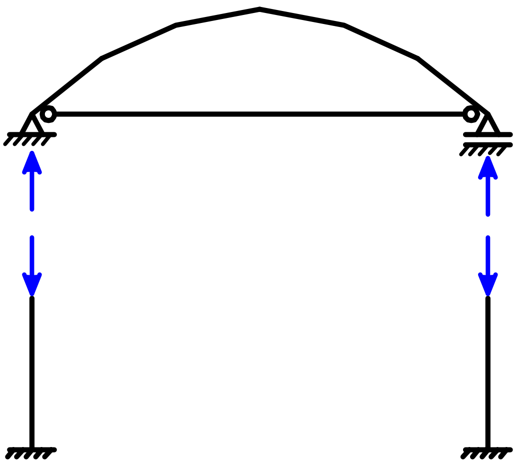

As1
As1
As2
As3
As4
As4
Sec. (1-1)
Sec. (2-2)
Sec. (3-3)
Sec. (4-4)
Sec. (5-5)
As1
2φ12
2φ12
As1
b
t
b
b
0.25
0.25
As4
2φ12
As4
0.25
0.25

Saw Tooth on Arch Girder.

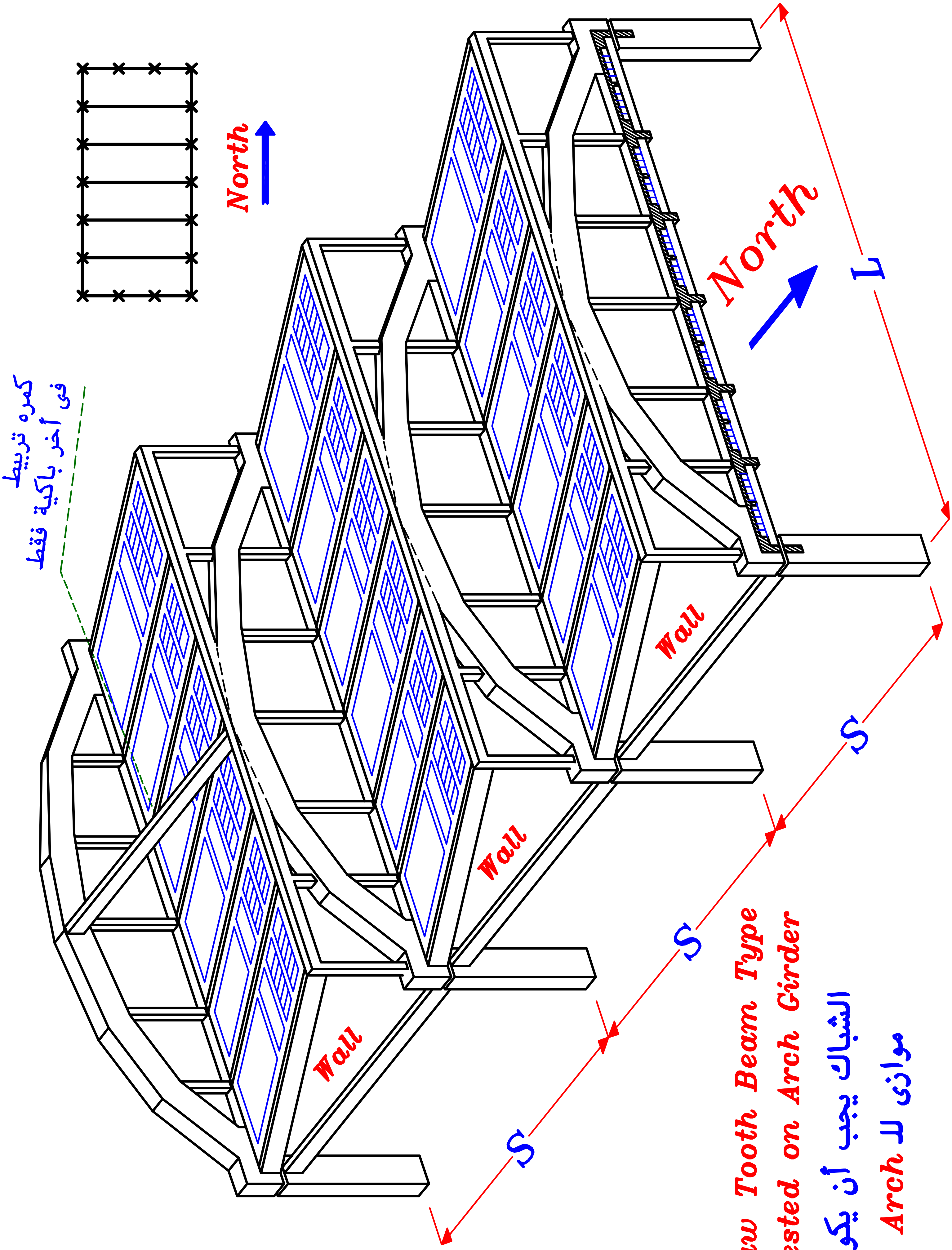


الشباك يجب أن يكون موازى لل Arch Girder
الشباك يجب أن يكون رأسى .

* Span (L) = (20 → 40) m
* Height (h) = $\frac{L}{6 \to 8}$
* $t_{(Arch)} \simeq \frac{L}{20 \to 25}$
* Tie (b x b)
* Hanger (250 x 250)
* $t_{col.} = \frac{H}{8 \to 9}$

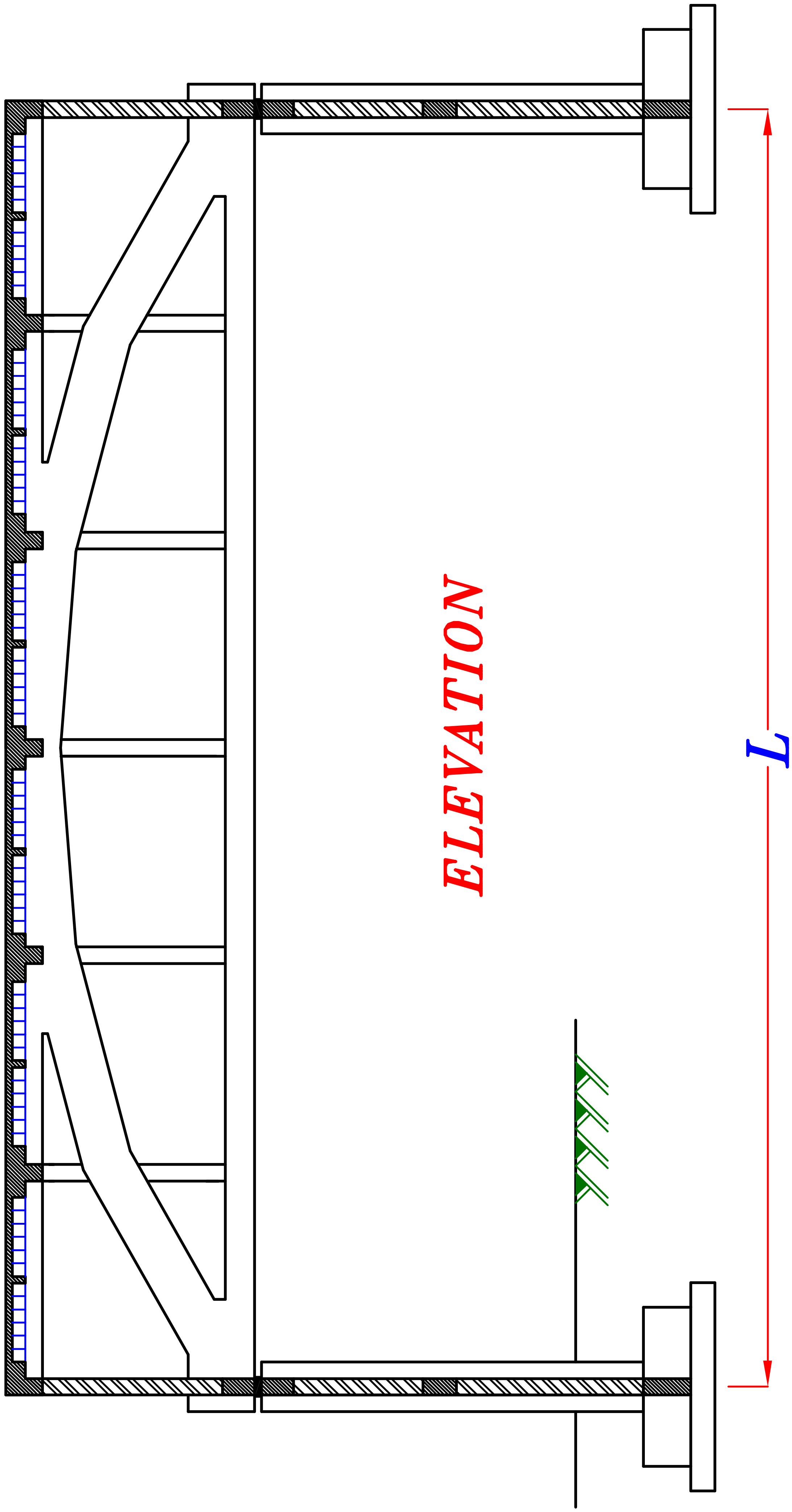


Statical System

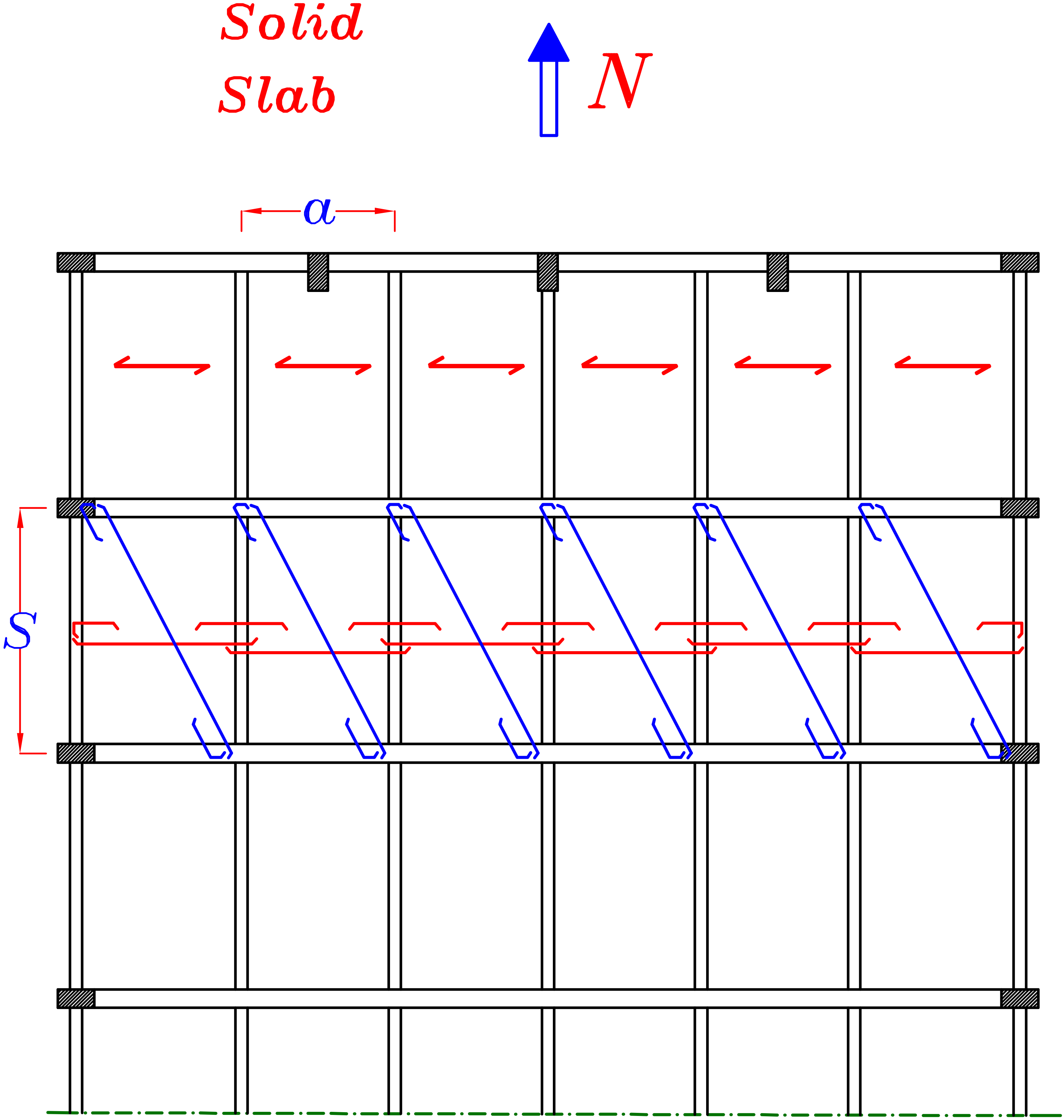


Saw Tooth Beam Type
Rested on Arch Girder

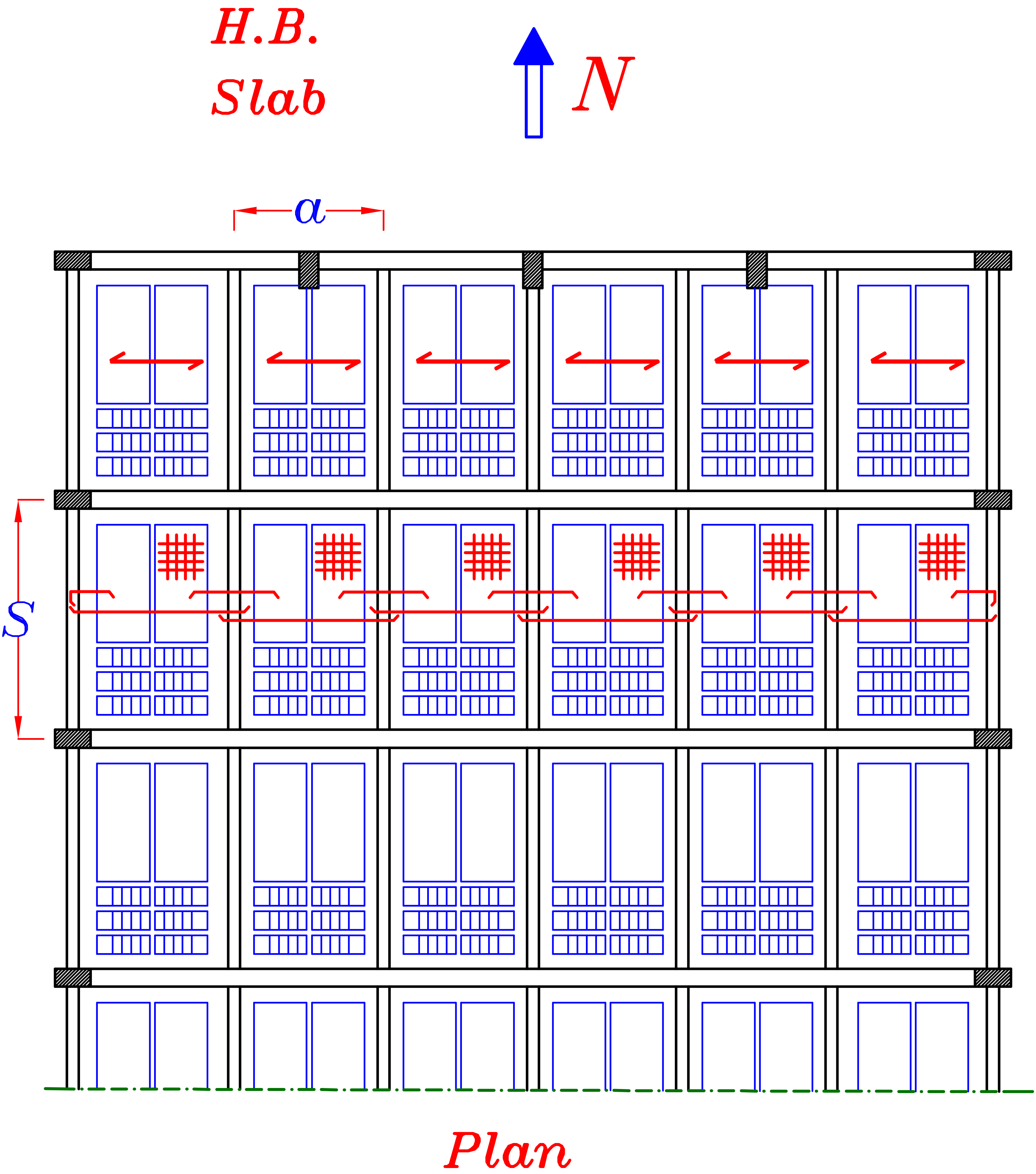
الشباك يجب أن يكون
موازى للـ **Arch**

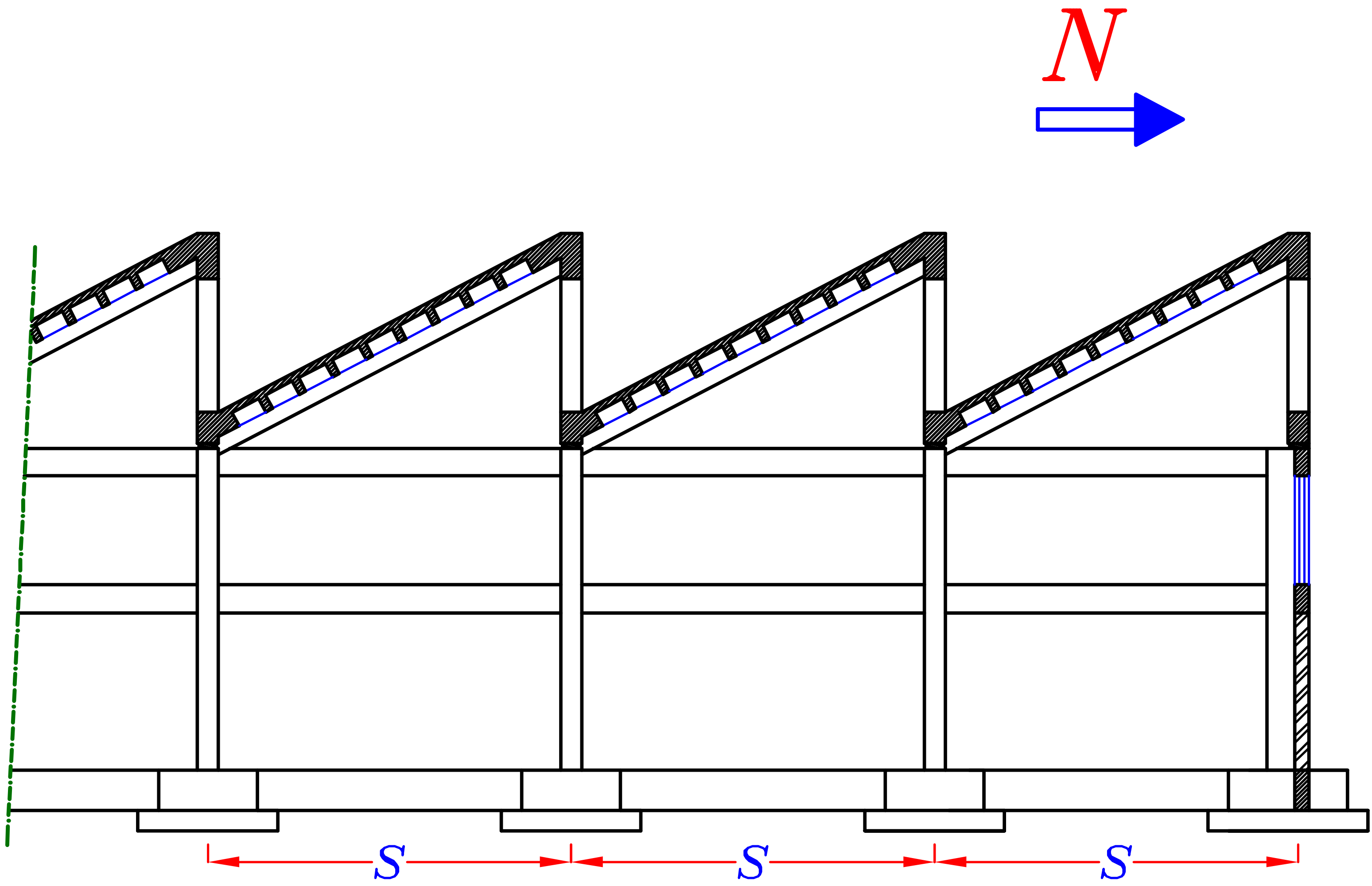

ELEVATION
L

Solid Slab



Plan

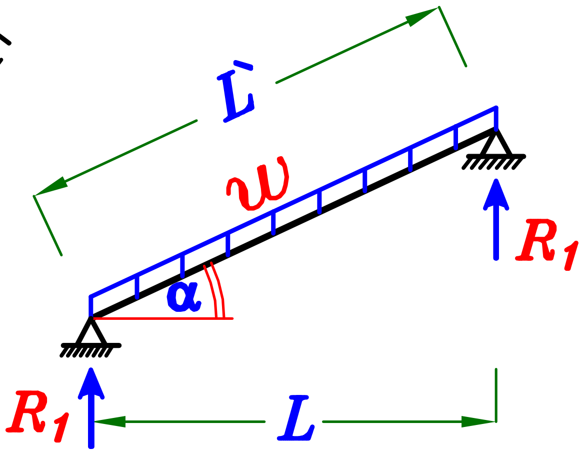

H.B.
Slab
N
a
S
Plan



SIDE VIEW

* Design the secondary Beam.

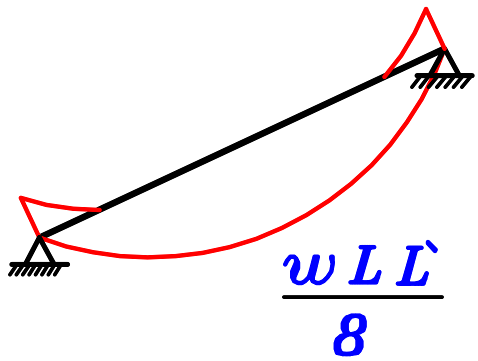
$$w = O.W._{(beam)} + w_s * a \quad kN\backslash m`$$



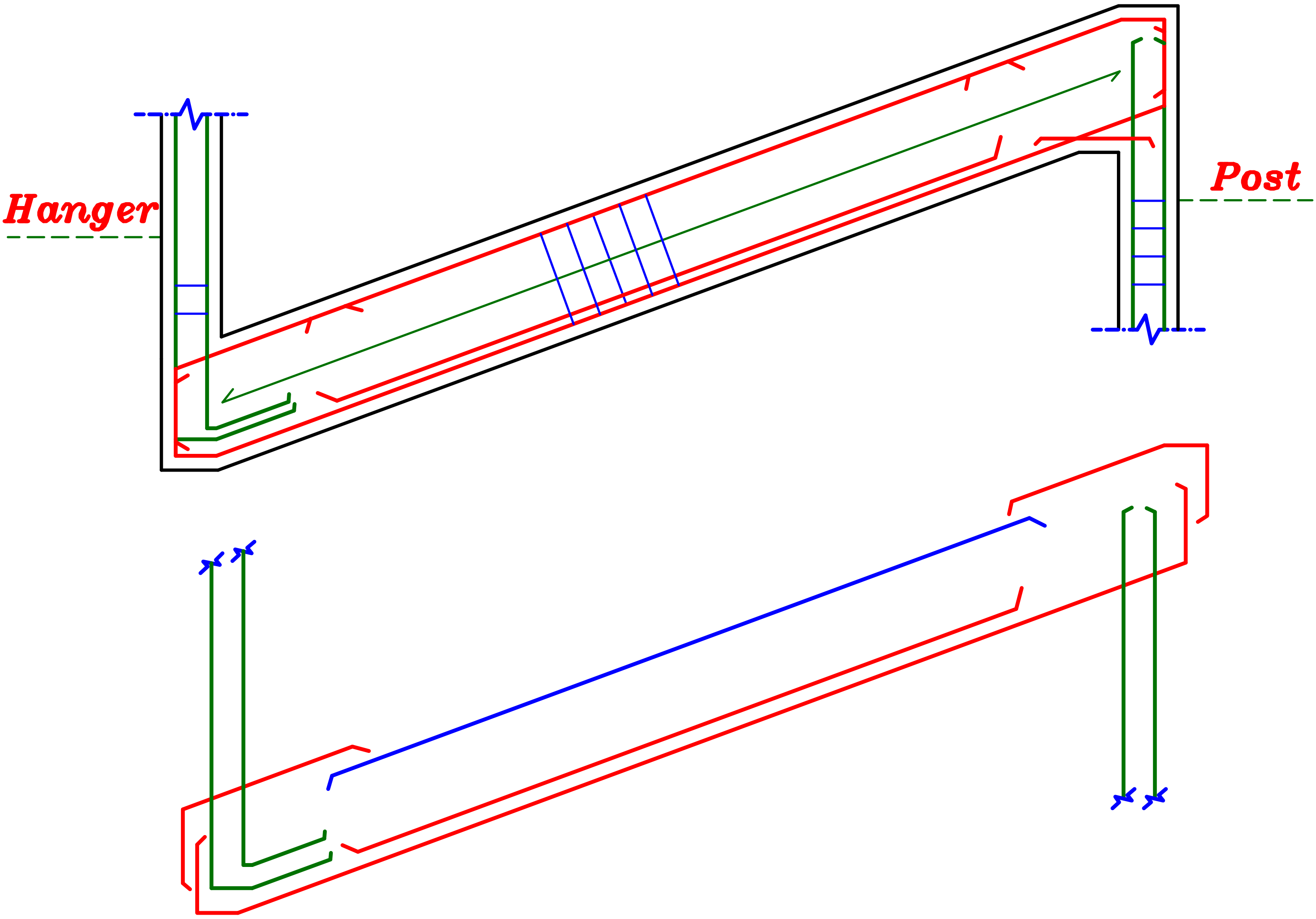
$$M = \frac{w\, L\, L`}{8}$$

Designed as T-sec.

$$R_1 = \frac{w\, L`}{2}$$



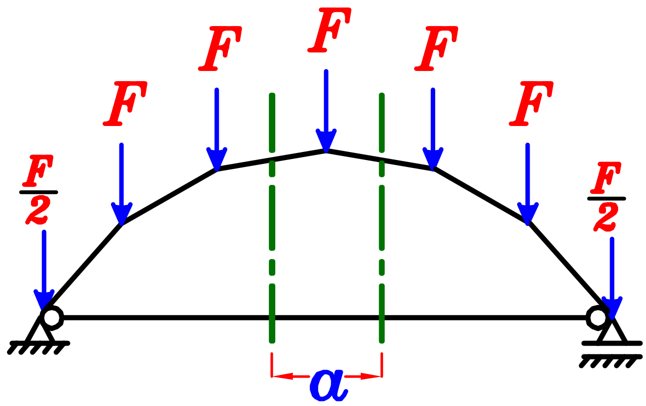
RFT. of the Beam in elevation.



* Loads on Arch girder.

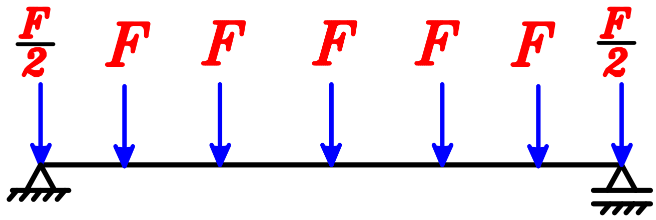
O.W. (Arch+ Tie+ Post+ Hanger+ Top beam) ≃ 17.5 kN\m` (U.L.)

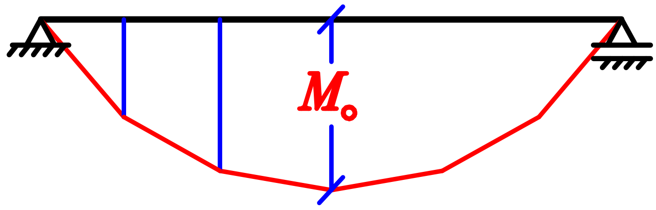
$$F = O.W. * a + R_1 + R_1$$



* Draw B.M.D.

Get M_o



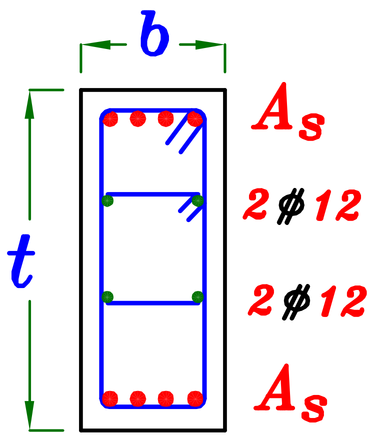


* Design the Arch Girder.

$M = 0.05\ M_o$

$P = 0.95\ \dfrac{M_o}{h}$

} Using I.D.

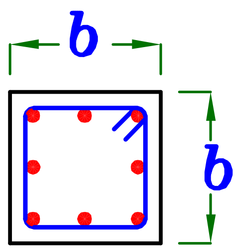


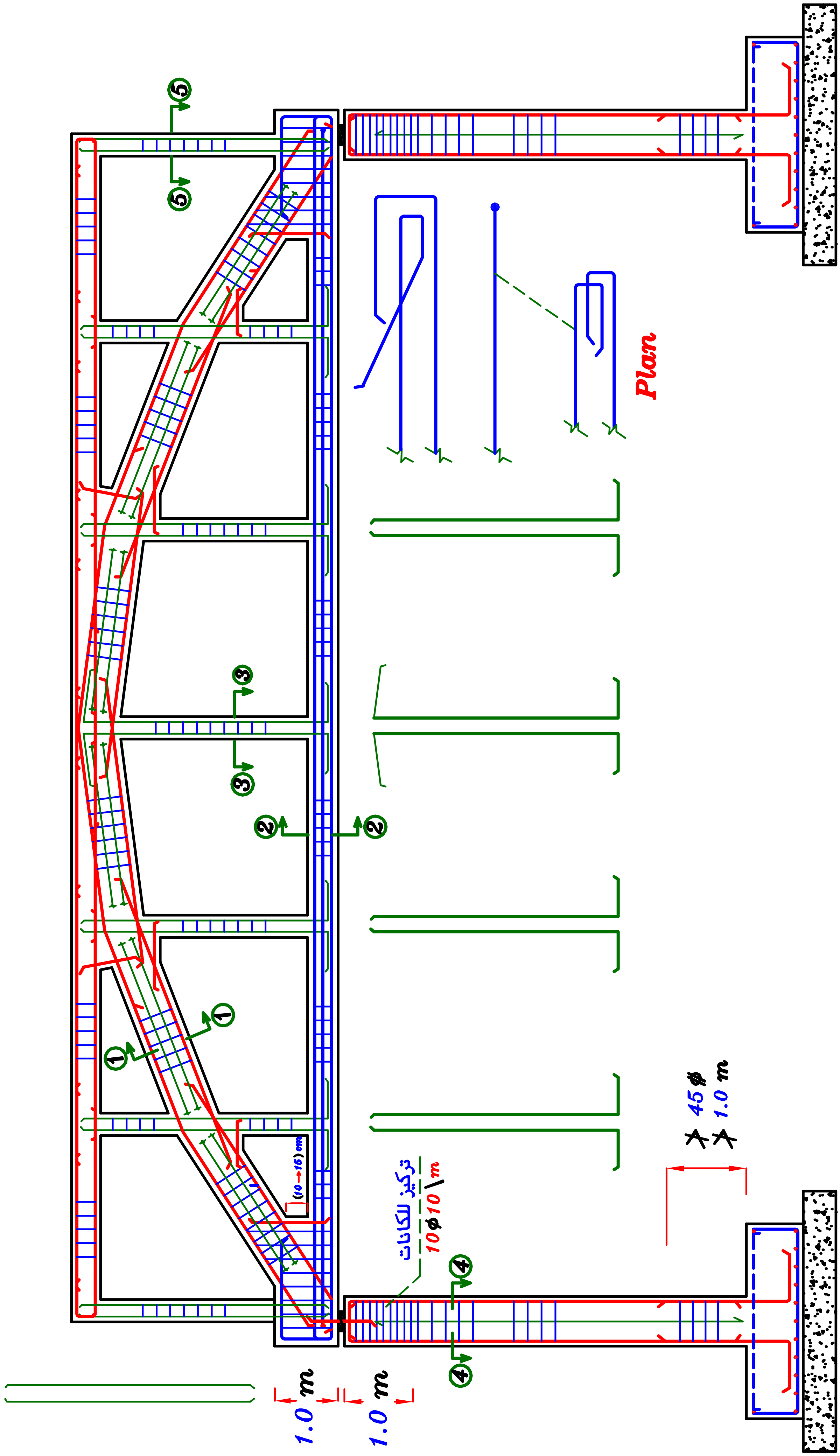
* Design the Tie.

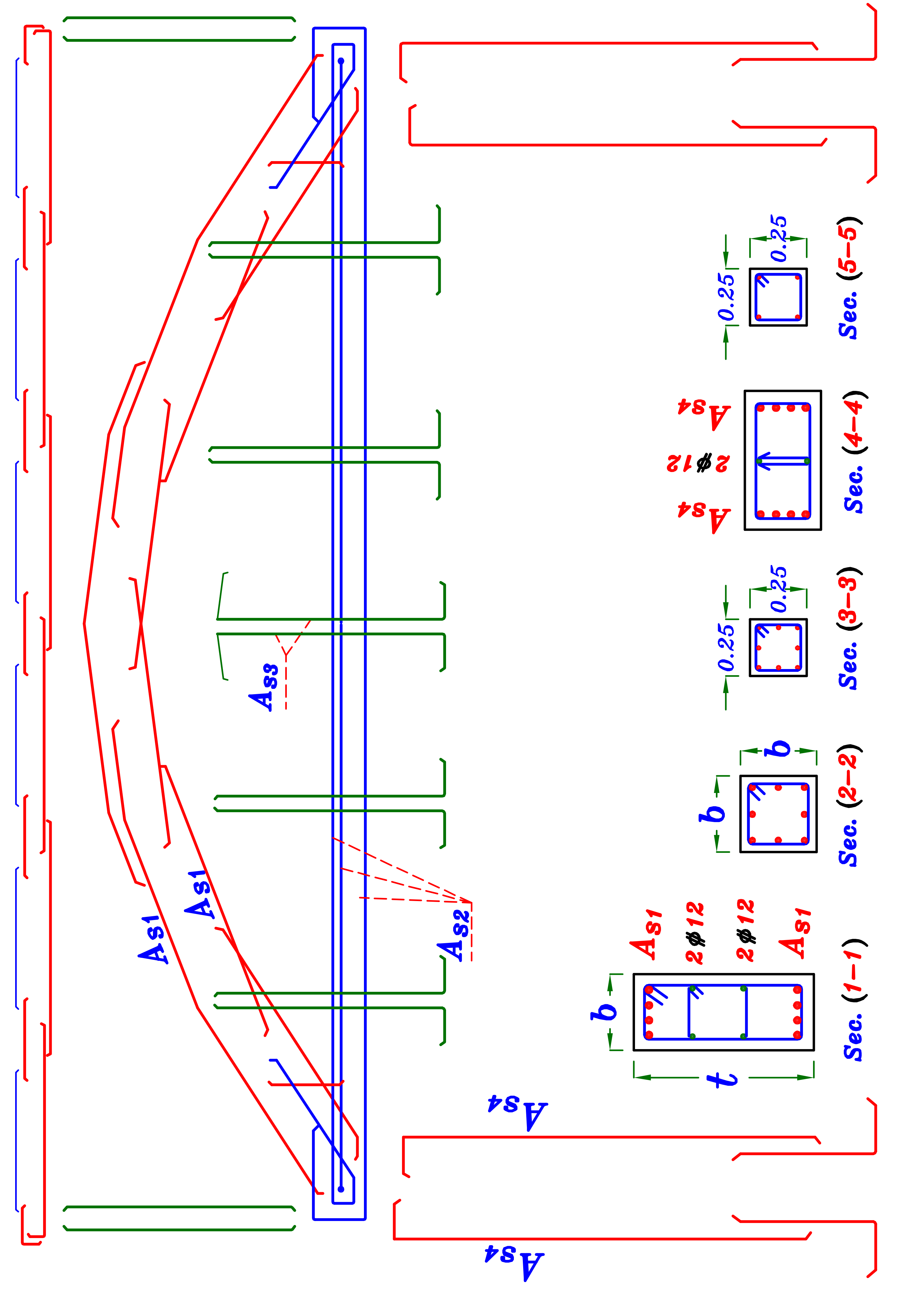
$$T = 0.95\ \frac{M_o}{h}$$

$$A_s = \frac{T}{F_y \backslash \gamma_s} = (\text{Total area of steel})$$

$$A_c = (b * b)$$

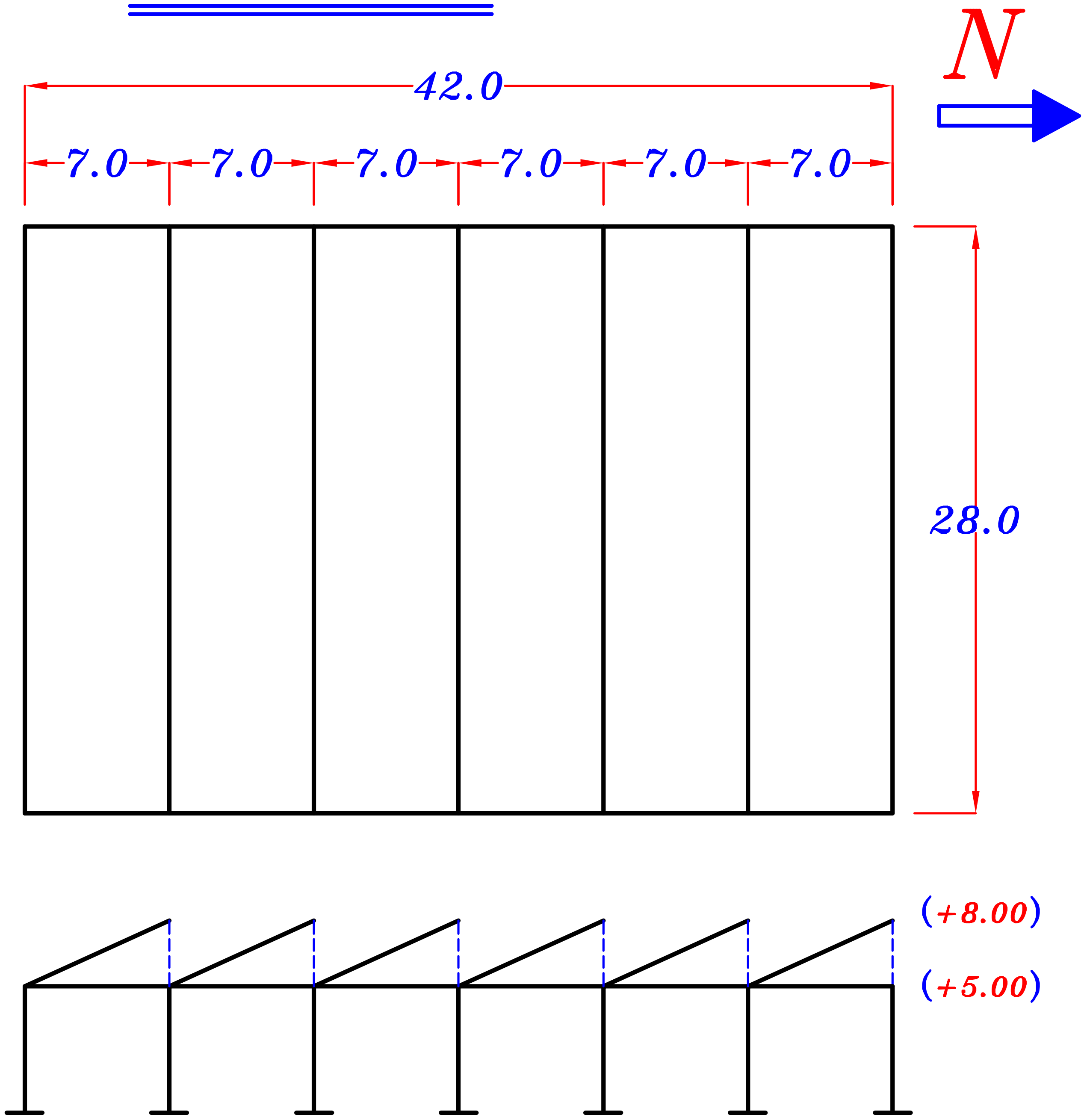



Plan
1.0 m
1.0 m
45 ϕ
1.0 m
تركيز للكانات
10ϕ10\m
(10→15) cm
1
2
3
4
5



As1
As1
As2
As3
As4
As4
Sec. (1-1)
b
t
As1
2ϕ12
2ϕ12
As1
Sec. (2-2)
b
b
Sec. (3-3)
0.25
0.25
Sec. (4-4)
As4
2ϕ12
As4
Sec. (5-5)
0.25
0.25

Example.

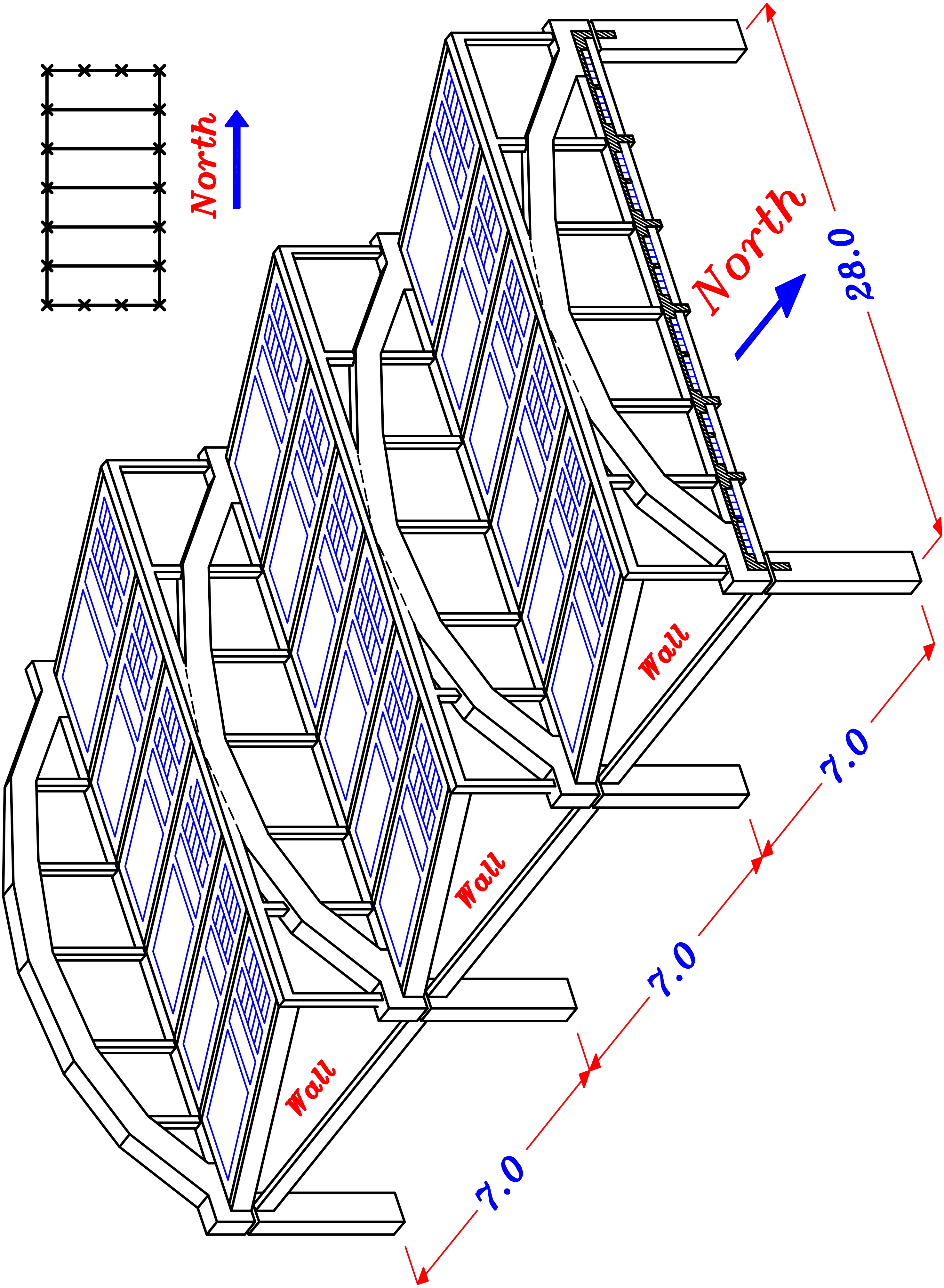


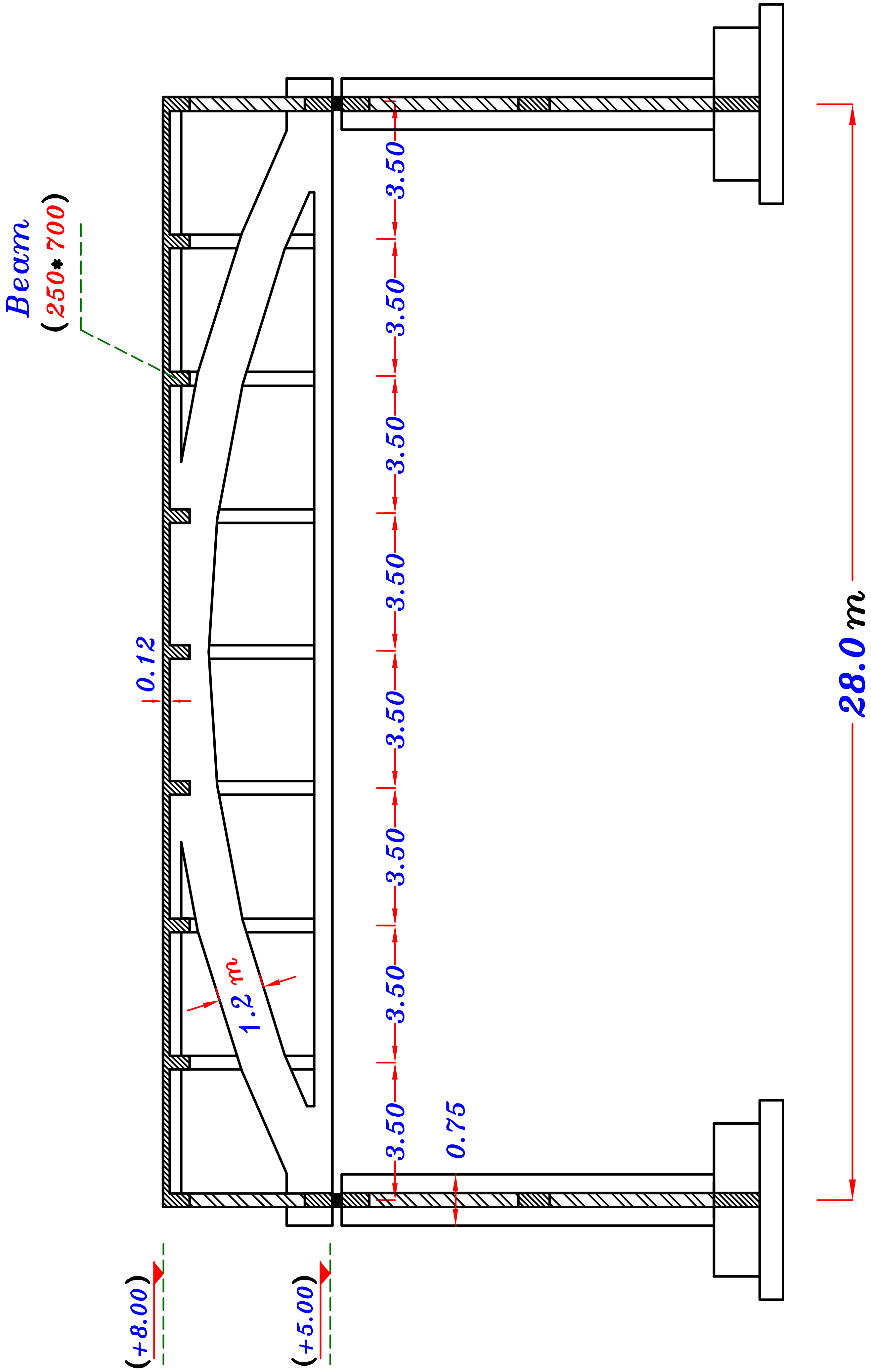
* $F_{cu} = 25 \ N\backslash mm^2$ $F_y = 360 \ N\backslash mm^2$

* Total loads (D.L.+L.L.) of the saw tooth roof are 8.0 $kN\backslash m^2$ H.P.

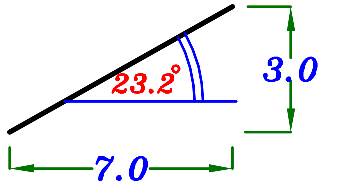
① Suggest a statical system to cover the area and draw Structural plan , Sectional elevation. showing all concrete dimensions.

② Design the main supporting elements, and draw Details of RFT. in elevation and cross sections.


North
North
28.0
7.0
7.0
7.0
Wall
Wall
Wall

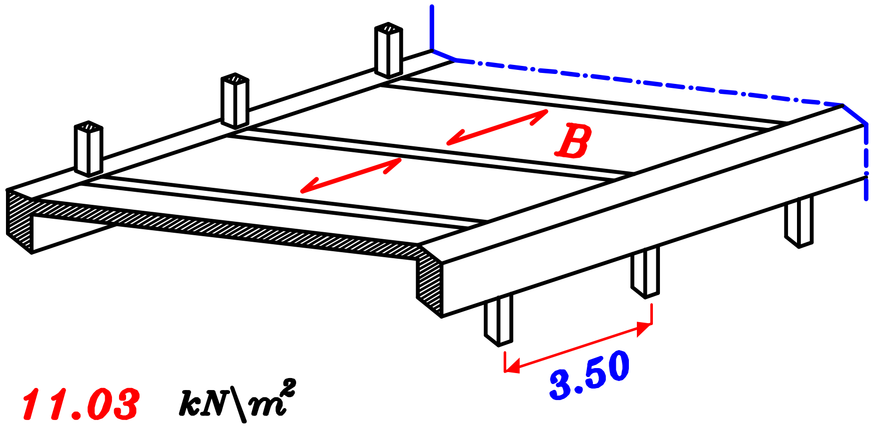

Beam
(250*700)
0.12
3.50
3.50
3.50
3.50
3.50
3.50
3.50
3.50
28.0 m
1.2 m
0.75
(+8.00)
(+5.00)

③ Design the Saw Tooth Slab.



* Loads on the Slab. (One Way Slab)

$t_s = \frac{3500}{30} = 116\ mm$ Take $\boxed{t_s = 120\ mm}$



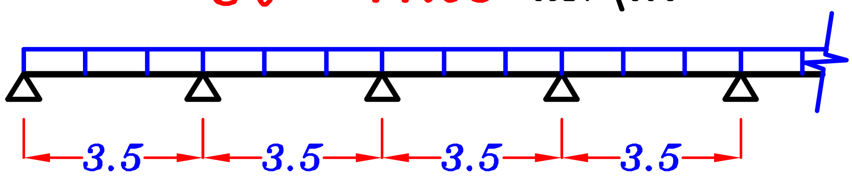
$(w_s)_{working} = 8.0\ kN\backslash m^2$ H.P.

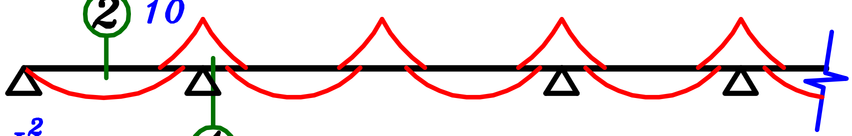
$(w_s)_{U.L.} = 1.5 * 8.0$

$= 12.0\ kN\backslash m^2$ H.P.

$(w_{si}) = 12.0 * \cos 23.2^\circ = 11.03\ kN\backslash m^2$

$w_{si} = 11.03\ kN\backslash m`$





* Design the Slab.

شريحه افقيه فى بلاطه مائله

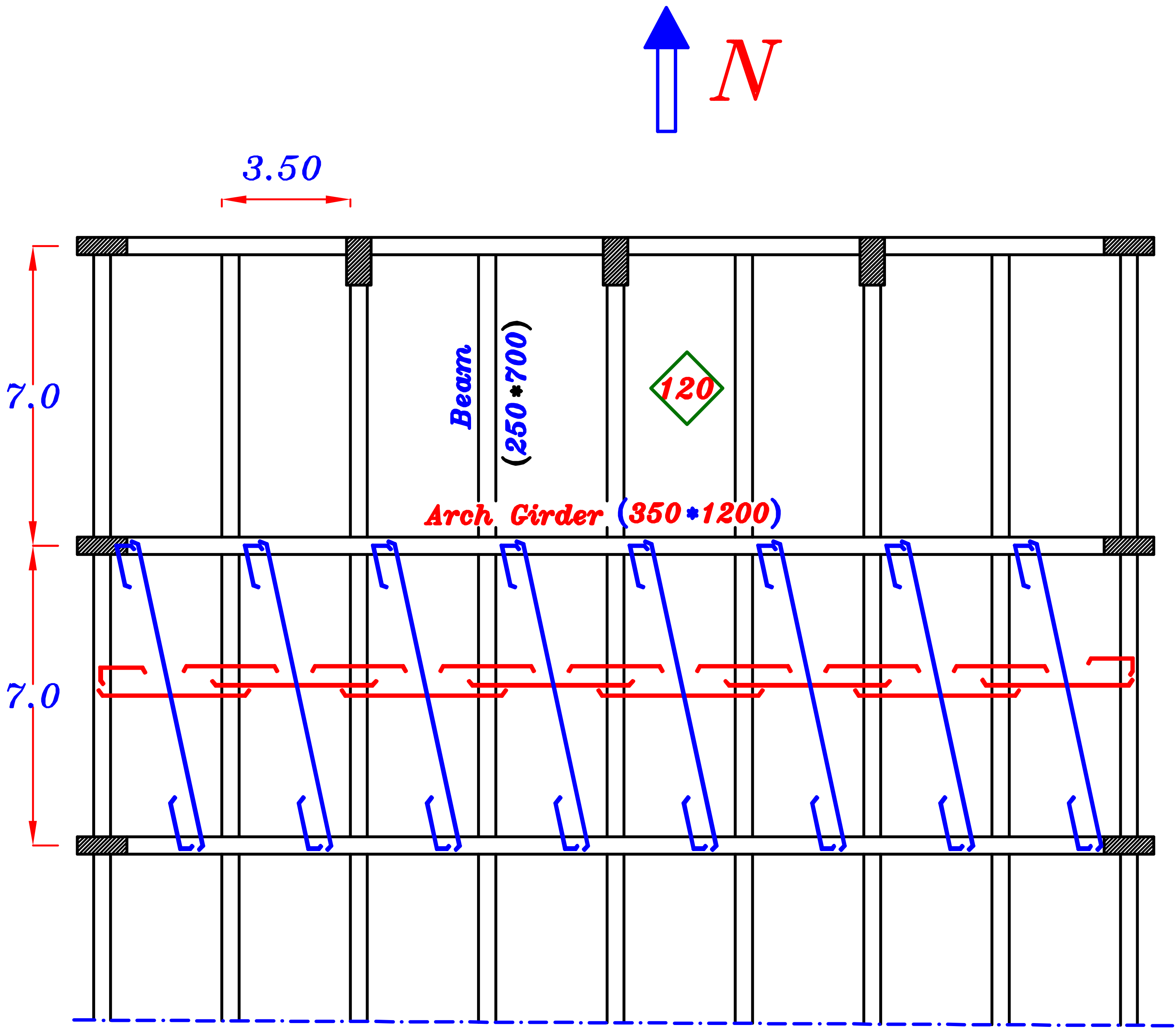
$M_{des.} = M \cos \alpha$

$= 13.51 * \cos 23.2^\circ = 12.40\ kN.m\backslash m`$

Sec. ① $M_{U.L.} = 12.4\ kN.m\backslash m`$, $t_s = 120\ mm$, $d = 100\ mm$

$100 = C_1 \sqrt{\frac{12.40 * 10^6}{25 * 1000}} \longrightarrow C_1 = 4.49 \longrightarrow J = 0.819$

$A_s = \frac{12.40 * 10^6}{0.819 * 360 * 100} = 420.56\ mm^2\backslash m$ $6 \phi 10 \backslash m`$

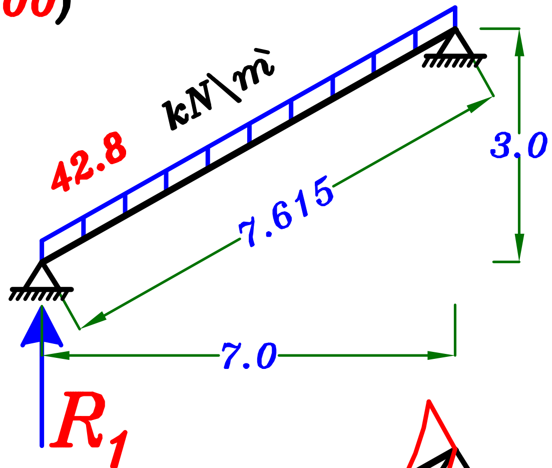


Plan

* Design the Beam. B (250*700)

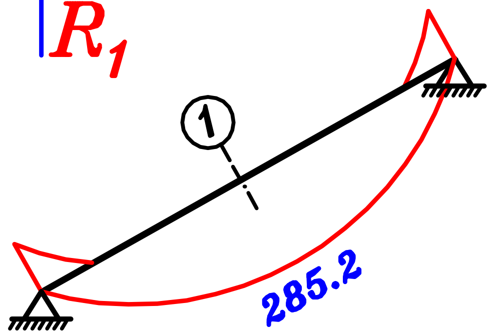
$w = O.W._{(beam)} + w_s * a \quad kN\backslash m'$

$w = 4.20 + 11.03 * 3.5 = 42.8 \quad kN\backslash m'$



$M = \dfrac{w L L'}{8} = 285.2 \quad kN.m$

$R_1 = \dfrac{w L'}{2} = 162.96 \quad kN$



Sec. 1

$M_{U.L.} = 285.2 \quad kN.m \qquad$ T-Sec.

$$B = \left\{ \begin{array}{lll} C.L.-C.L. & = 3.5\ m = 3500 & mm \\ 16\, t_s + b & = 16 * 140 + 250 = 2490 & mm \\ K \dfrac{L}{5} + b & = 1.0 * \dfrac{7615}{5} + 250 = 1773 & mm \end{array} \right\} \qquad \boxed{B = 1773 \quad mm}$$

$$\therefore 650 = C_1 \sqrt{\frac{285.2 * 10^6}{25 \quad * 1773}} \longrightarrow C_1 = 8.10 \longrightarrow J = 0.826$$

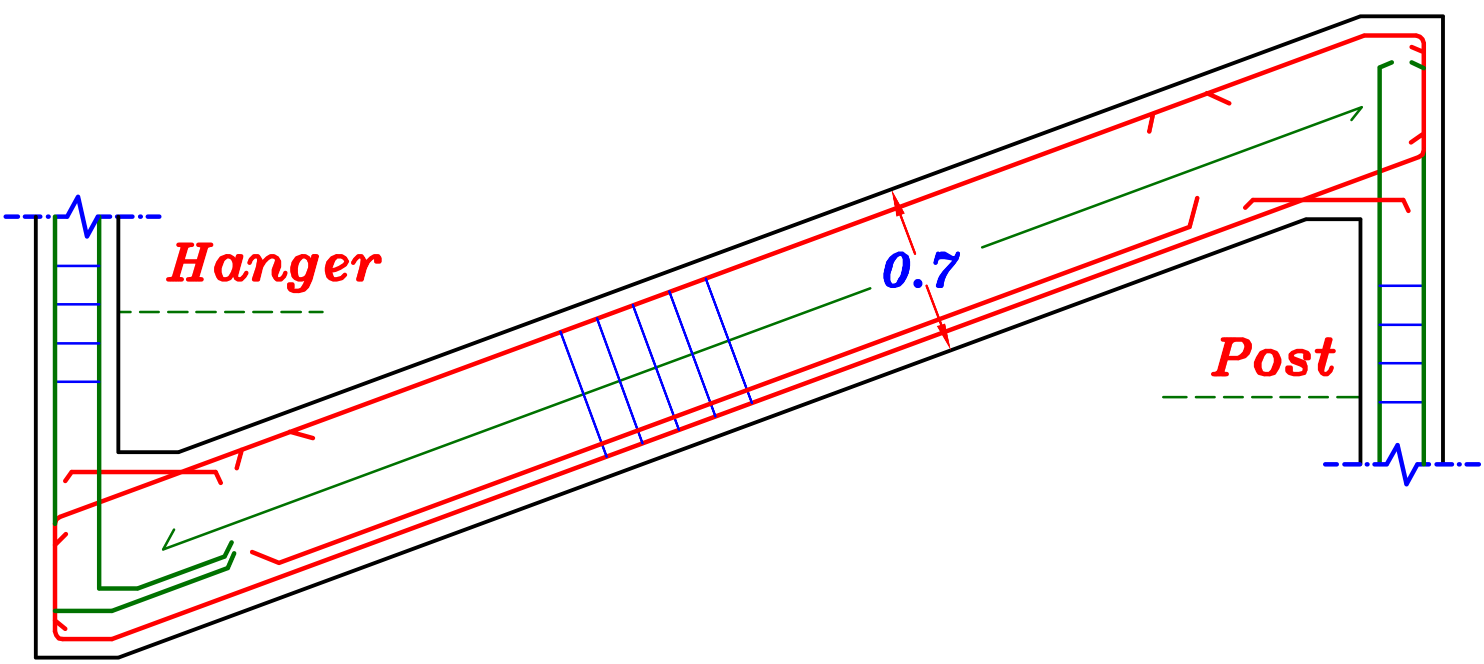
$$\therefore A_s = \frac{285.2 * 10^6}{0.826 * 360 \quad * 650} = 1475.5 \ mm^2$$

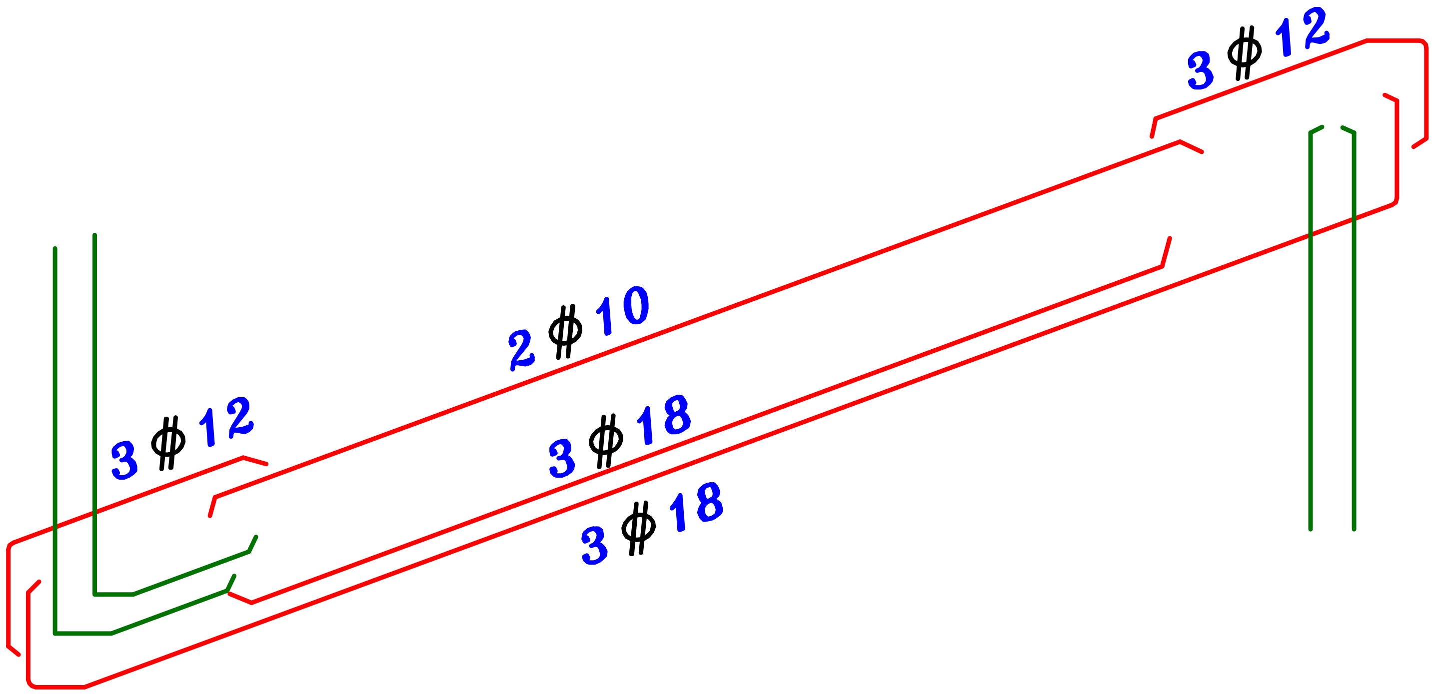
Check $A_{s\,min.}$ $\qquad A_{s_{req.}} = 1475.5 \ mm^2$

$$\mu_{min.} b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 650 = 507.8 \ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.} b\ d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1475.5 \ mm^2 \qquad$ 6 ϕ18

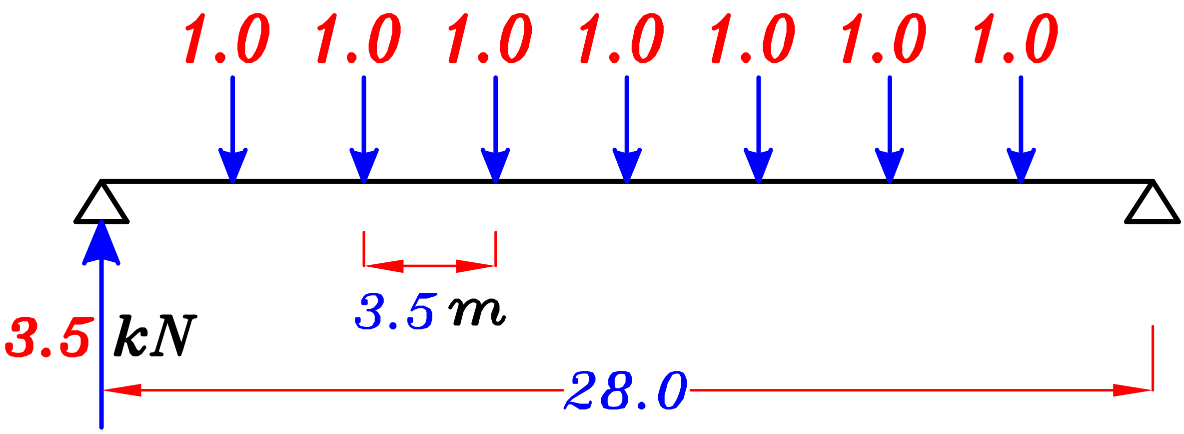
$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{18 + 25} = 5.23 = 5.0 \text{ bars}$$

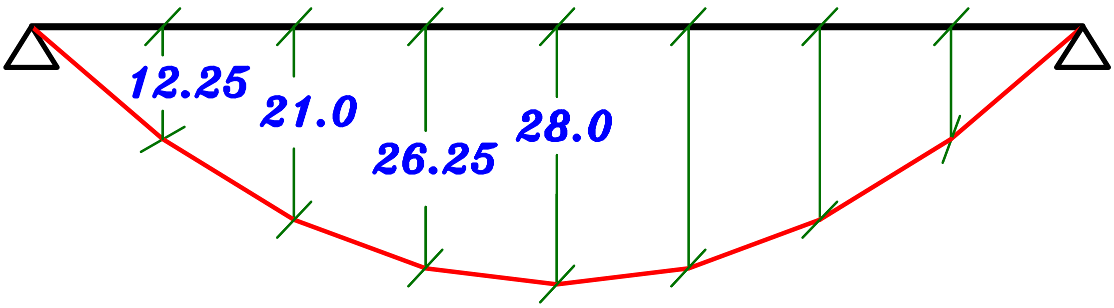

Hanger
0.7
Post


3 ф 12
2 ф 10
3 ф 12
3 ф 18
3 ф 18

4 Drawing Arch Girder.

Height of the Arch Girder



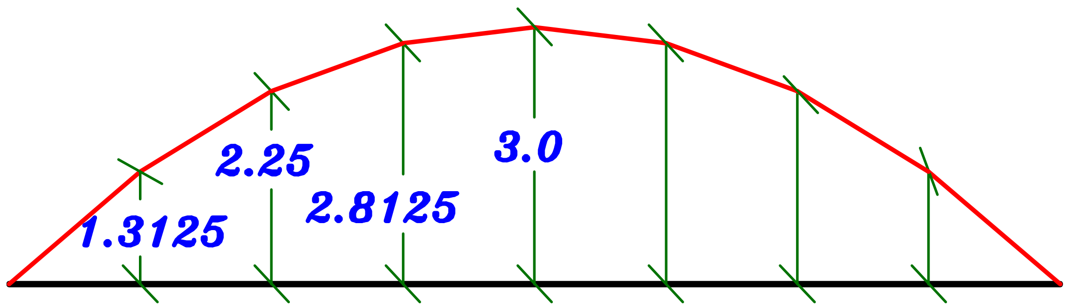


$M_o = 28.0 \quad kN.m \longrightarrow h_o = 3.0 \ m$

$M_1 = 26.25 \quad kN.m \longrightarrow h_1 = 2.8125 \ m$

$M_2 = 21.0 \quad kN.m \longrightarrow h_2 = 2.25 \ m$

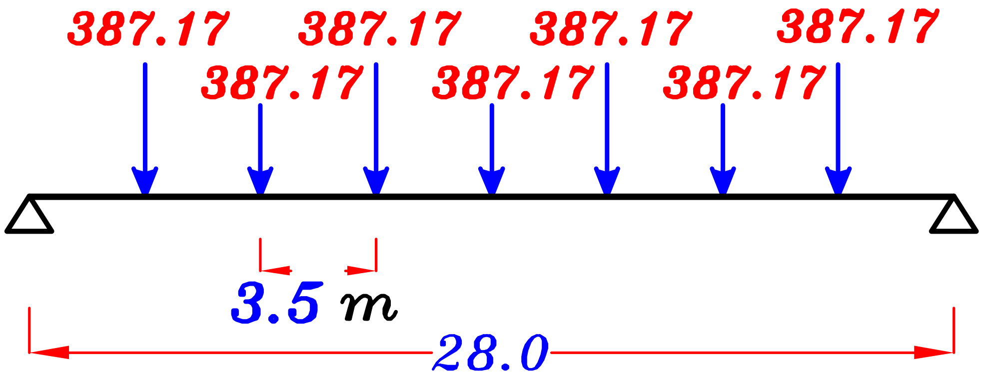
$M_3 = 12.25 \quad kN.m \longrightarrow h_3 = 1.3125 \ m$

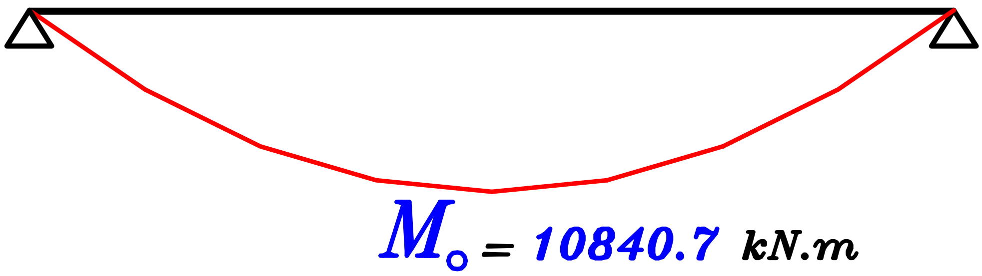


Loads on the Arch Girder.

Take $o.w.(Arch) = 17.5 \ kN\backslash m'$ (U.L.)

$$f = 2\,R_1 + o.w.(Arch) * a = 2\,(162.96) + 17.5\,(3.5) = 387.17 \ kN$$

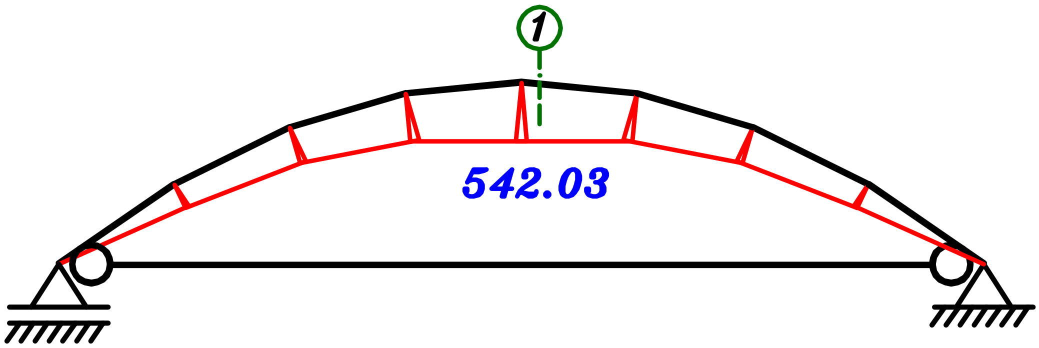




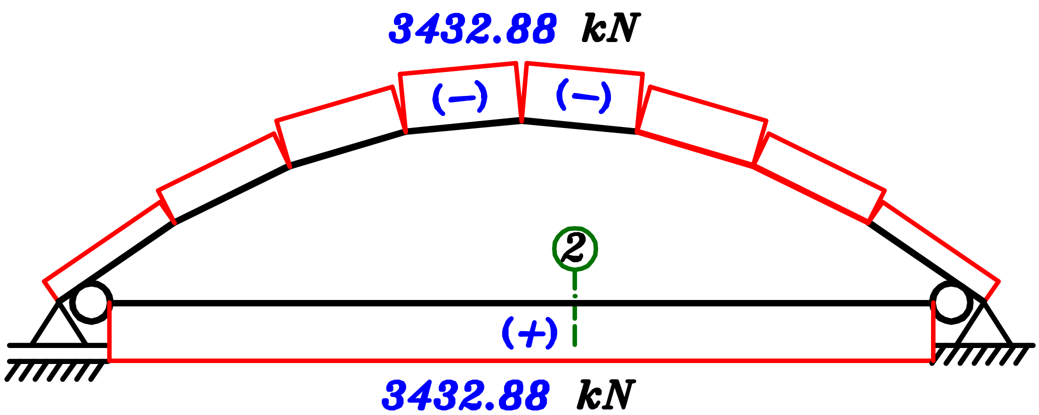
$$P = T = 0.95 \ \frac{M_o}{h} = 0.95 * \frac{10840.7}{3.0} = 3432.88 \ kN$$

$$M = 0.05 \ M_o = 0.05\,(10840.7) = 542.03 \ kN.m$$

B.M.D.



N.F.D.



* Design of Arch Girder.

Sec. 1 $b = 0.35\ m$, $t = 1.20\ m$

$$P = 0.95\ \frac{M_o}{h} = 0.95 * \frac{10840.7}{3.0} = 3432.88\ kN$$

$$M = 0.05\ M_o = 0.05\left(10840.7\right) = 542.03\ kN.m$$

$$e = \frac{M}{P} = \frac{542.03}{3432.88} = 0.158\ m \quad \therefore \frac{e}{t} = \frac{0.158}{1.2} = 0.13 < 0.5 \xrightarrow{use} I.D.$$

$$\zeta = \frac{1.2 - 0.2}{1.2} = 0.83 \xrightarrow{use} \text{ECCS Design Aids Page } 4\text{–}24$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\ b\ t} &= \frac{3432.88 * 10^3}{25 * 350 * 1200} = 0.327 \\ \frac{M_U}{F_{cu}\ b\ t^2} &= \frac{542.03 * 10^6}{25 * 350 * 1200^2} = 0.043 \end{aligned}\right\} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 25 * 10^{-4} = 2.5 * 10^{-3}$$

$$A_s = A_s` = \mu * b * t = 2.5 * 10^{-3} * 350 * 1200 = 1050\ mm^2$$

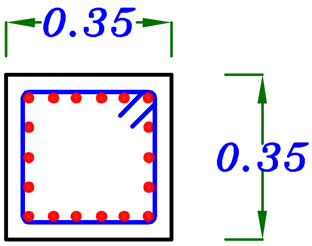
$$A_{s\,total} = A_s + A_s` = 2100\ mm^2$$

Check $A_{s\,min.} = \frac{0.8}{100} * b * t = \frac{0.8}{100} * 350 * 1200 = 3360\ mm^2 > A_{s\,total}$

$\therefore$ **Take** $A_s = A_s` = \frac{A_{s\,min.}}{2} = \frac{2520}{2} = 1680\ mm^2$ $5\phi22$

* Design of Tie.

Sec. ② (350 * 350)



$18\phi28$

Neglect o.w. of Tie.

$$T = 0.95\,\frac{M_o}{h} = 0.95 * \frac{10840.7}{3.0} = 3432.88 \text{ kN}$$

$$A_s = \frac{T}{F_y \backslash \delta_s} = \frac{3432.88 * 10^3}{360 \backslash 1.15} = 10966 \text{ mm}^2$$

$18\phi28$

* Design of the hangers. (350 * 350)

Take o.w.(hanger) = 3.50 kN (U.L.)

$$T = o.w.(hanger) + R_1$$

$$= 3.50 + 162.96 = 166.46 \text{ kN}$$

$$A_s = \frac{T}{F_y \backslash \delta_s} = \frac{166.46 * 10^3}{360 \backslash 1.15} = 531.7 \text{ mm}^2$$

$8\phi12$

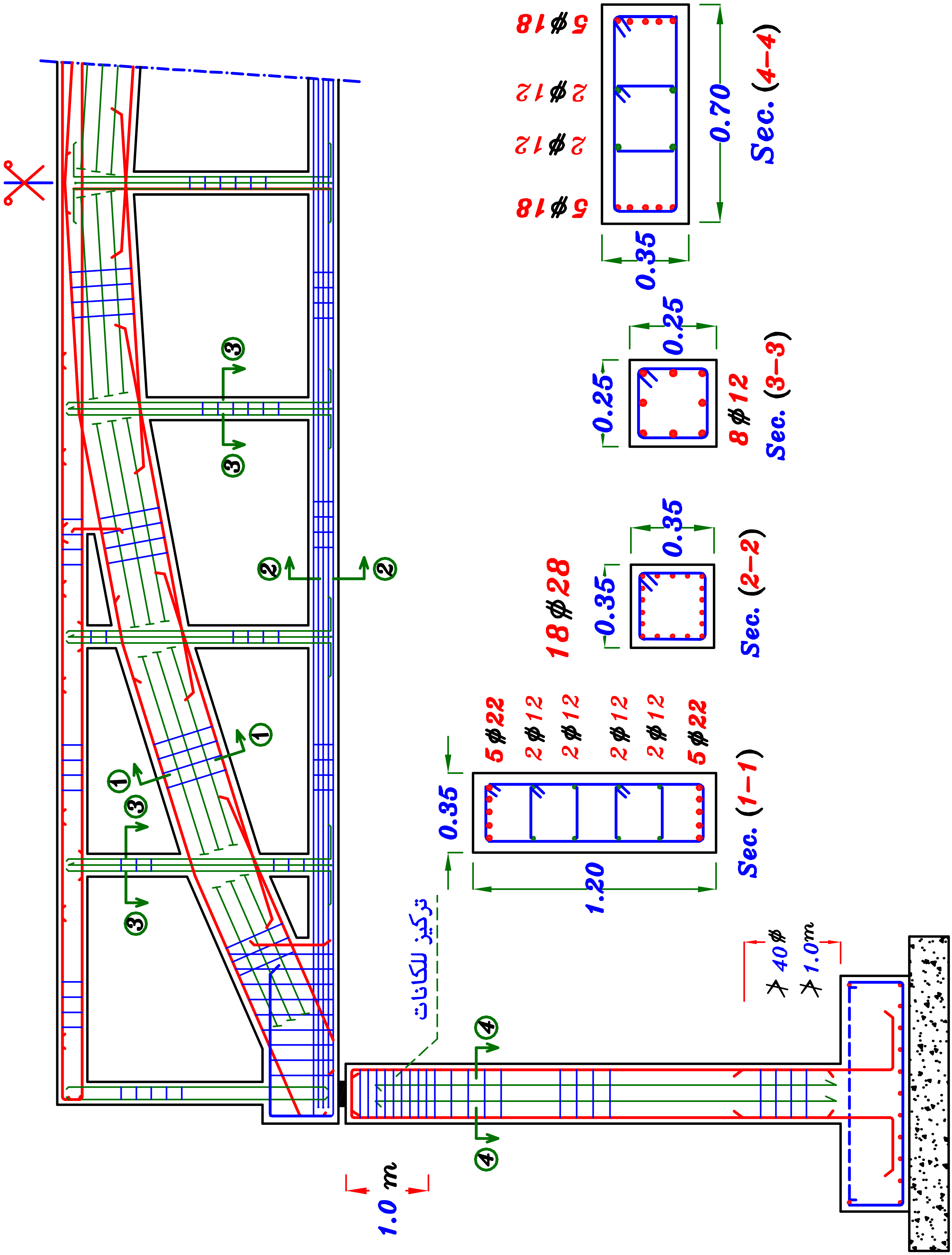
* Design the Post. (350 * 350)

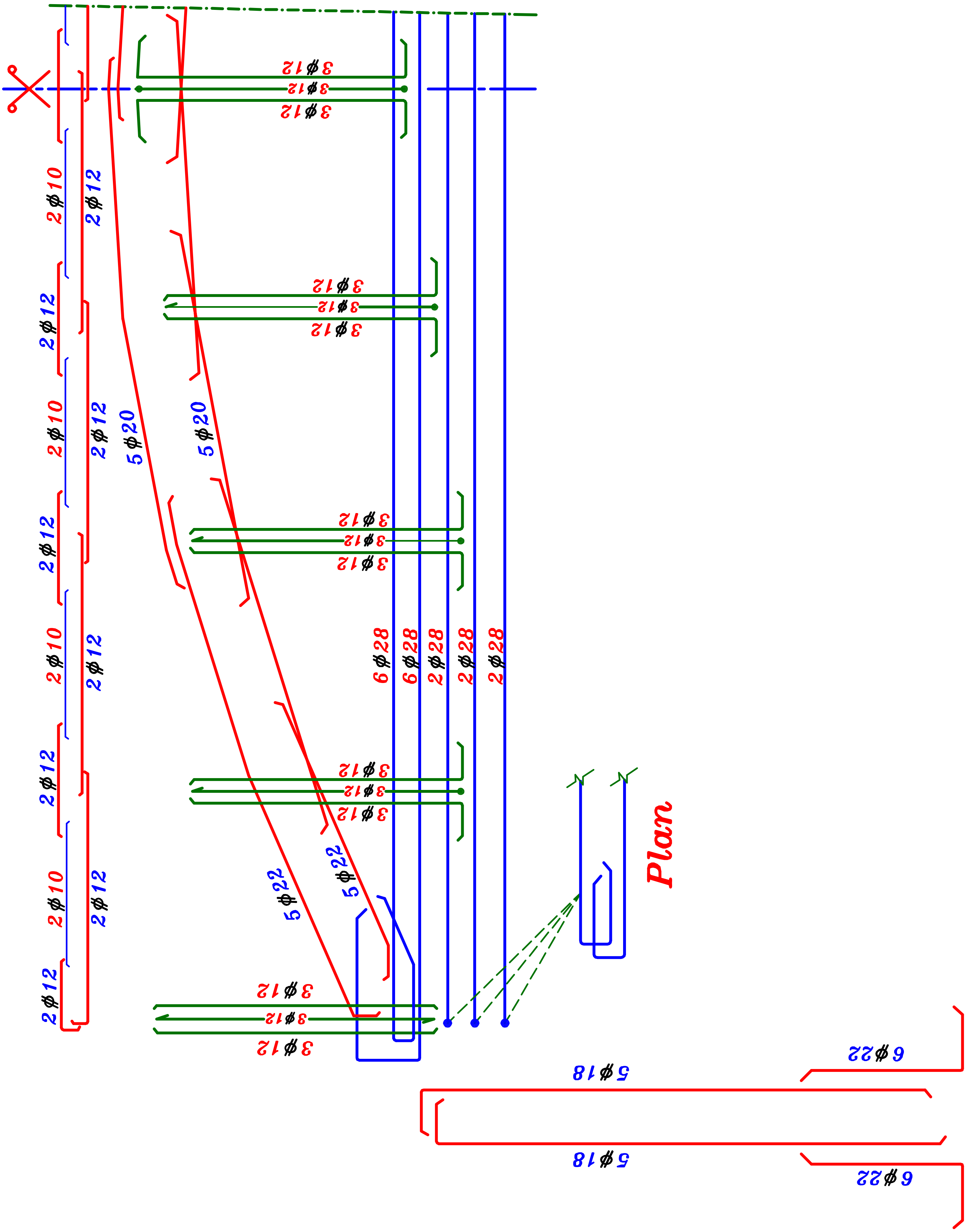
Take o.w.(Post) = 3.50 kN (U.L.)

$$P = o.w.(Post) + R_1$$

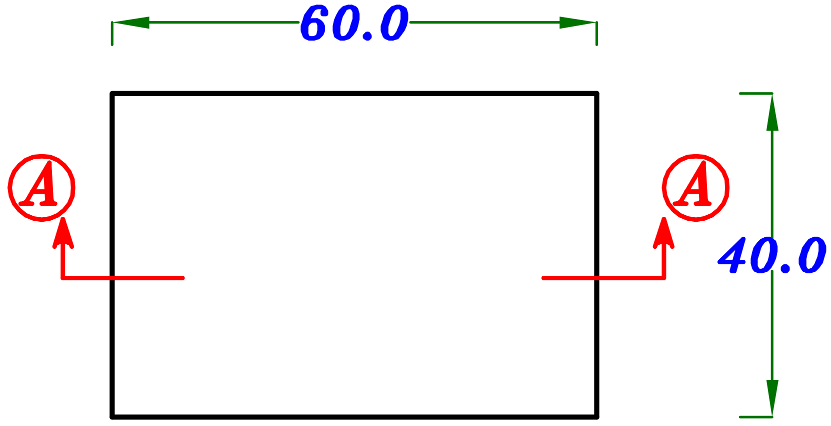
$$= 3.50 + 162.96 = 166.46 \text{ kN}$$

$$A_s = 8\phi12$$

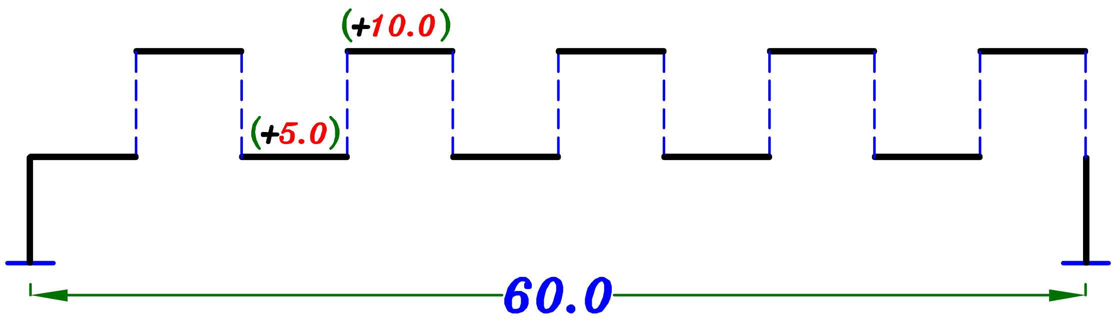

5Φ18
2Φ12
2Φ12
5Φ18
0.70
0.35
Sec. (4-4)
0.25
0.25
8Φ12
Sec. (3-3)
0.35
0.35
18Φ28
Sec. (2-2)
5Φ22
2Φ12
2Φ12
2Φ12
2Φ12
5Φ22
0.35
1.20
Sec. (1-1)
تركيز للكانات
40Φ
1.0m
1.0 m


Plan
3 ϕ 12
2 ϕ 10
2 ϕ 12
5 ϕ 20
5 ϕ 22
6 ϕ 28
2 ϕ 28
5 ϕ 18
6 ϕ 22

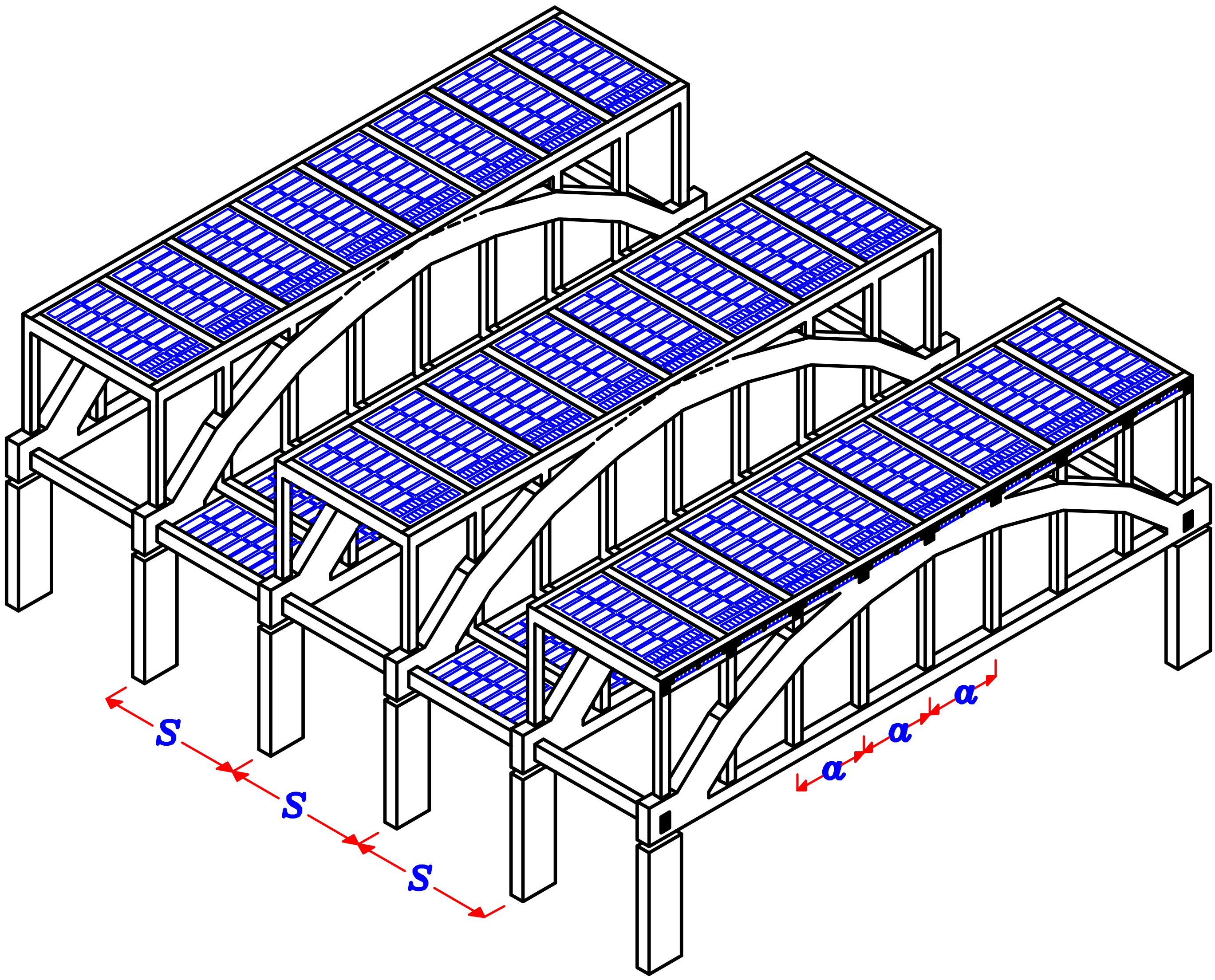
Example.

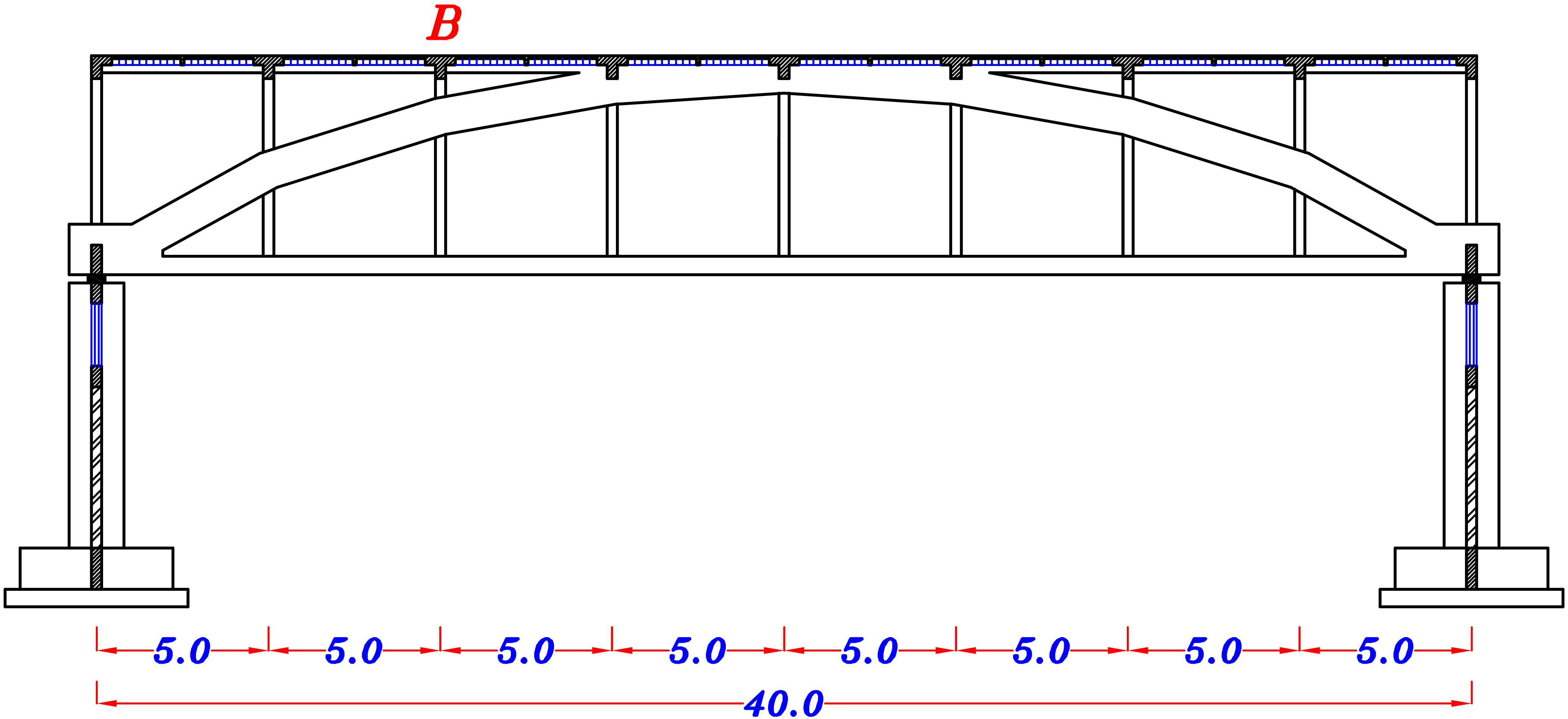


Sec. (A–A)



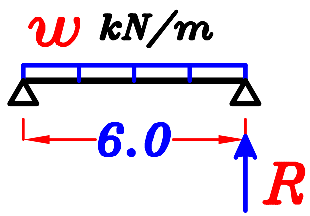
Choose a convenient Statical System and draw a sketch For an elevation Showing Concrete Dimensions.





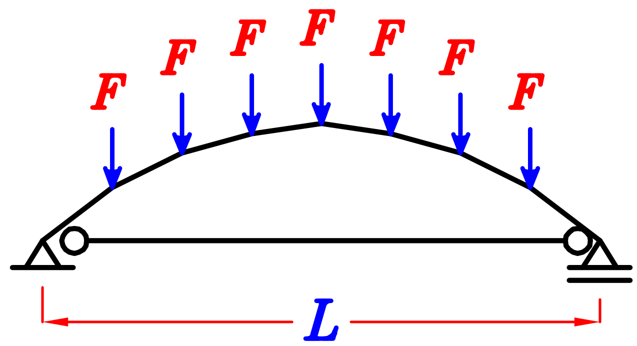
Beam B

$$w = o.w. + \left(\frac{w_{rib}}{S}\right) * a$$



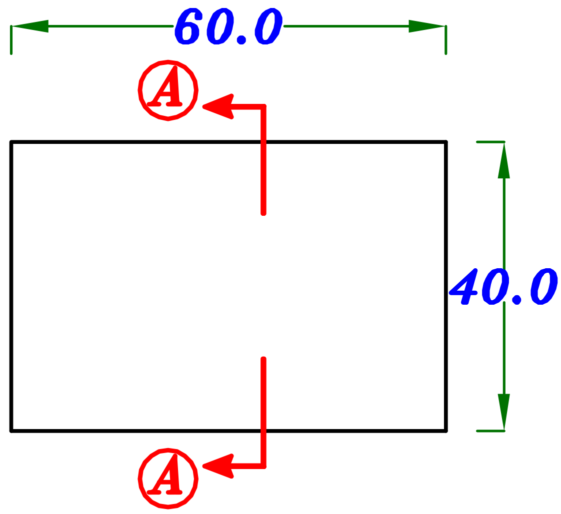
$$R = w * \frac{S}{2}$$

Arch Girder.

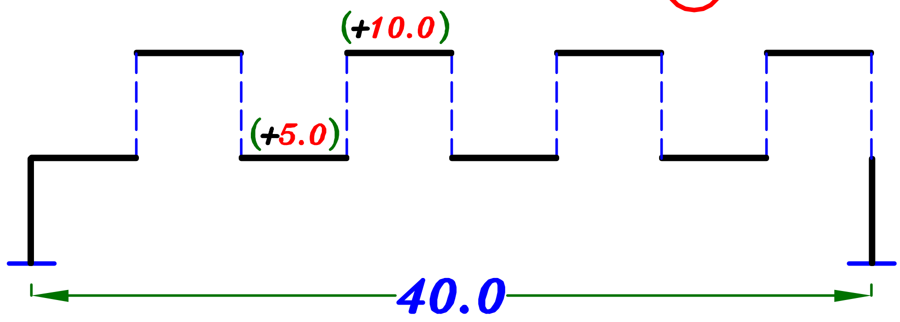


$$F = 2R + o.w. * a$$

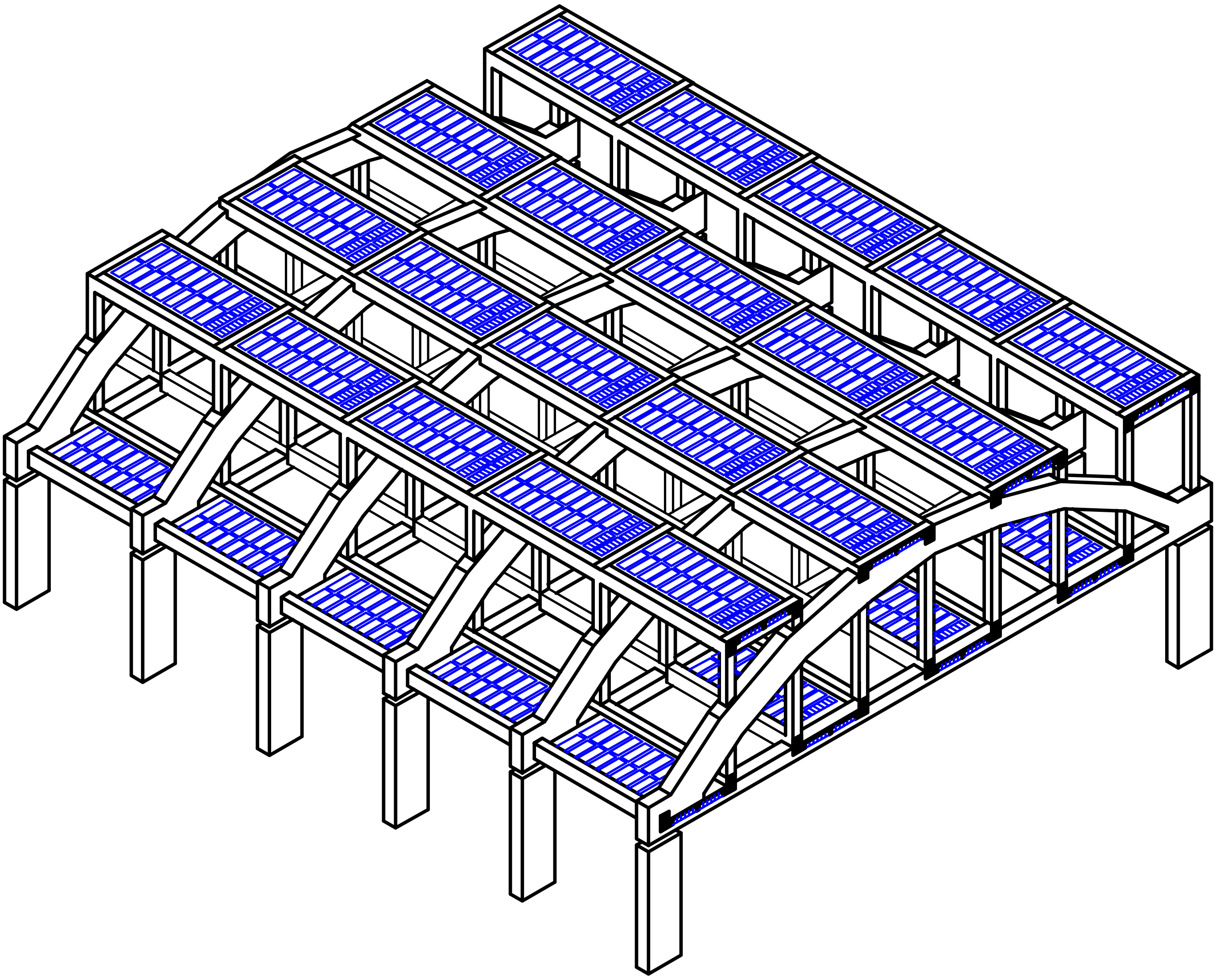
Example.

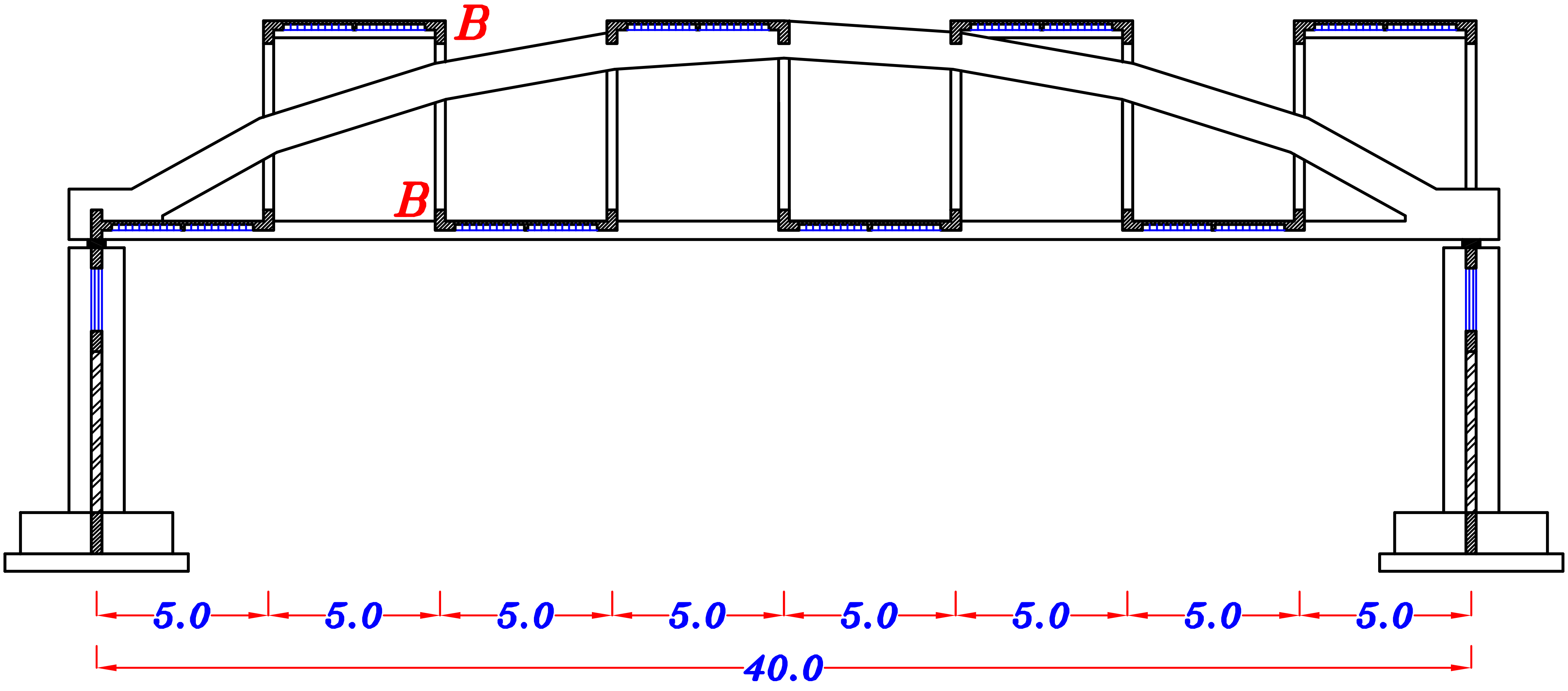


Sec. (A–A)



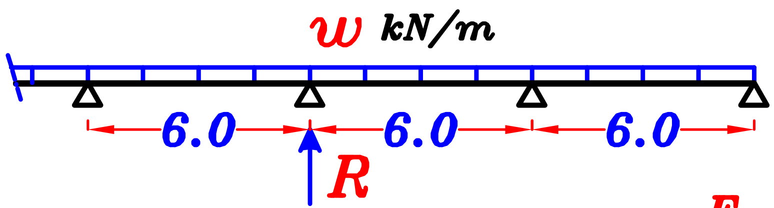
Choose a convenient Statical System and draw a sketch For an elevation Showing Concrete Dimensions.





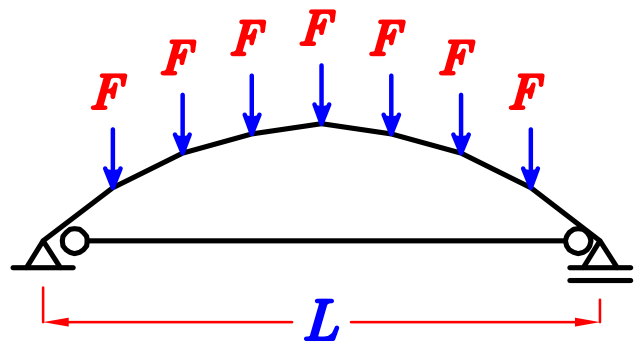
Beam B

$$w = o.w. + \left(\frac{w_{rib}}{S}\right) * \frac{a}{2}$$



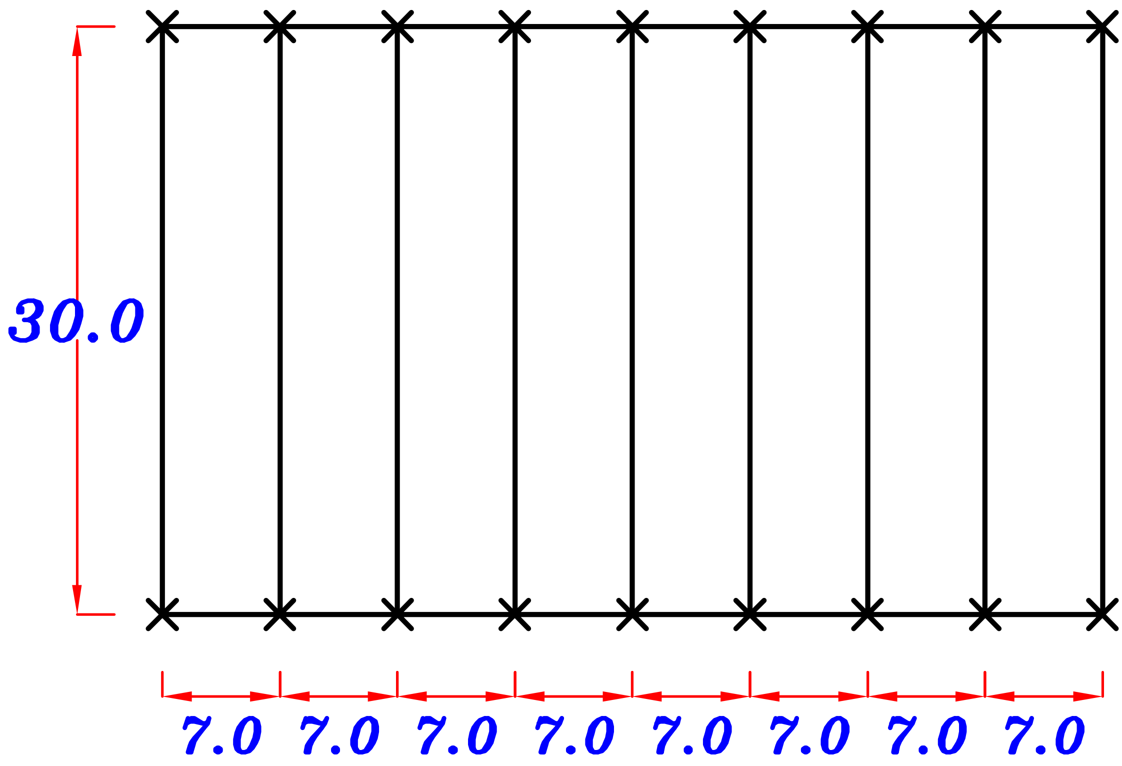
$$R = w * S$$

Arch Girder.



$$F = 2R + o.W. * a$$

Example.



$\boxed{L.L.+F.C. > 10 \; kN/m^2}$

Choose a convenient Statical System and draw a sketch For an elevation Showing Concrete Dimensions.

IF $\boxed{L.L.+F.C. > 10 \; kN/m^2}$ $\longrightarrow$ ***use Solid slabs.***

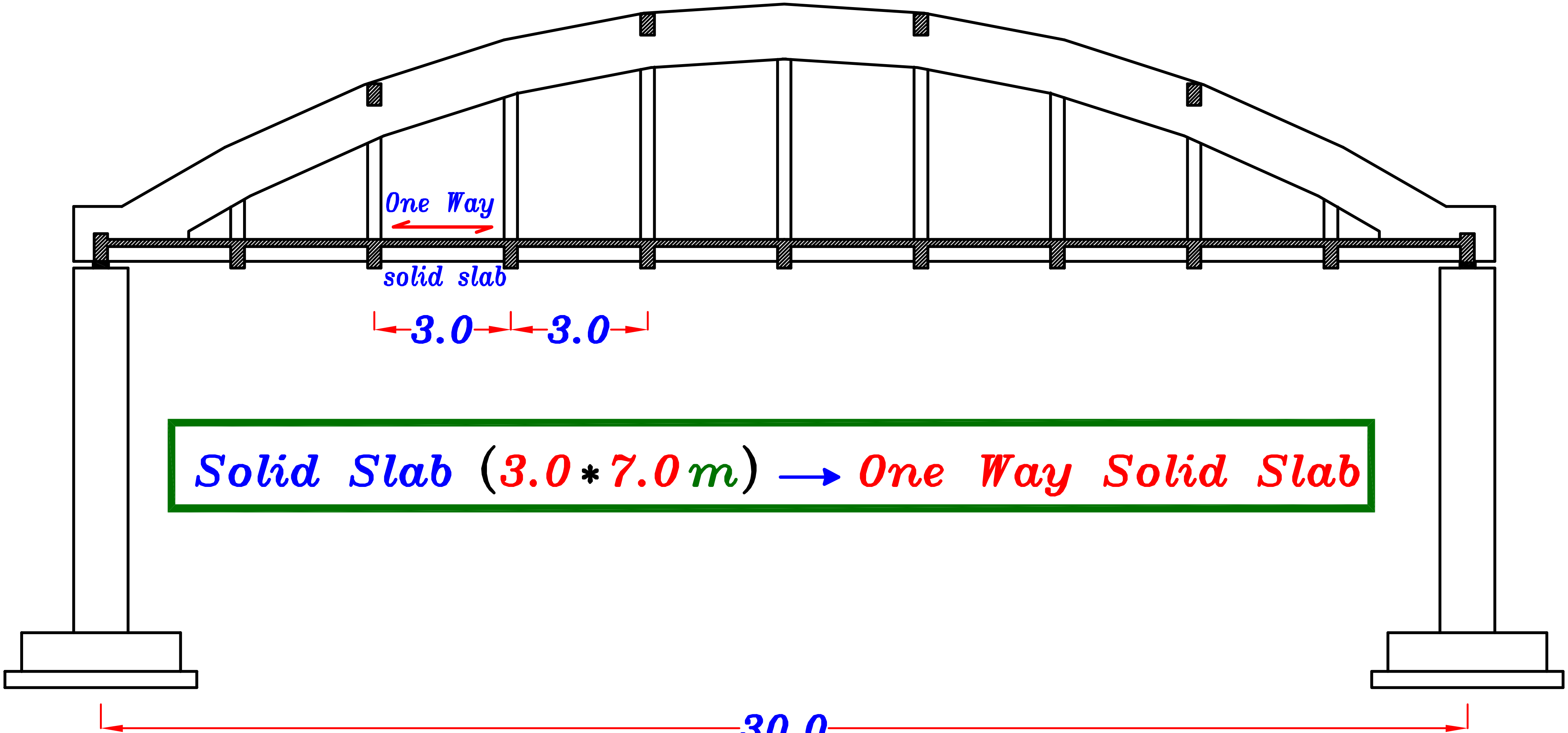
اذا كان مجموع $L.L.+F.C.$ أكبر من $10 \; kN/m^2$ يجب أن تكون البلاطه ***Solid Slab***

و لان البلاطه فى ال ***Arch Girder*** يجب أن تكون ***one way*** فى اتجاه الكمرات

يوجد حلان :

١ـ أن نزيد من عدد ال ***segments*** لل ***Arch Girder*** بحيث تكون $a < \frac{S}{2}$

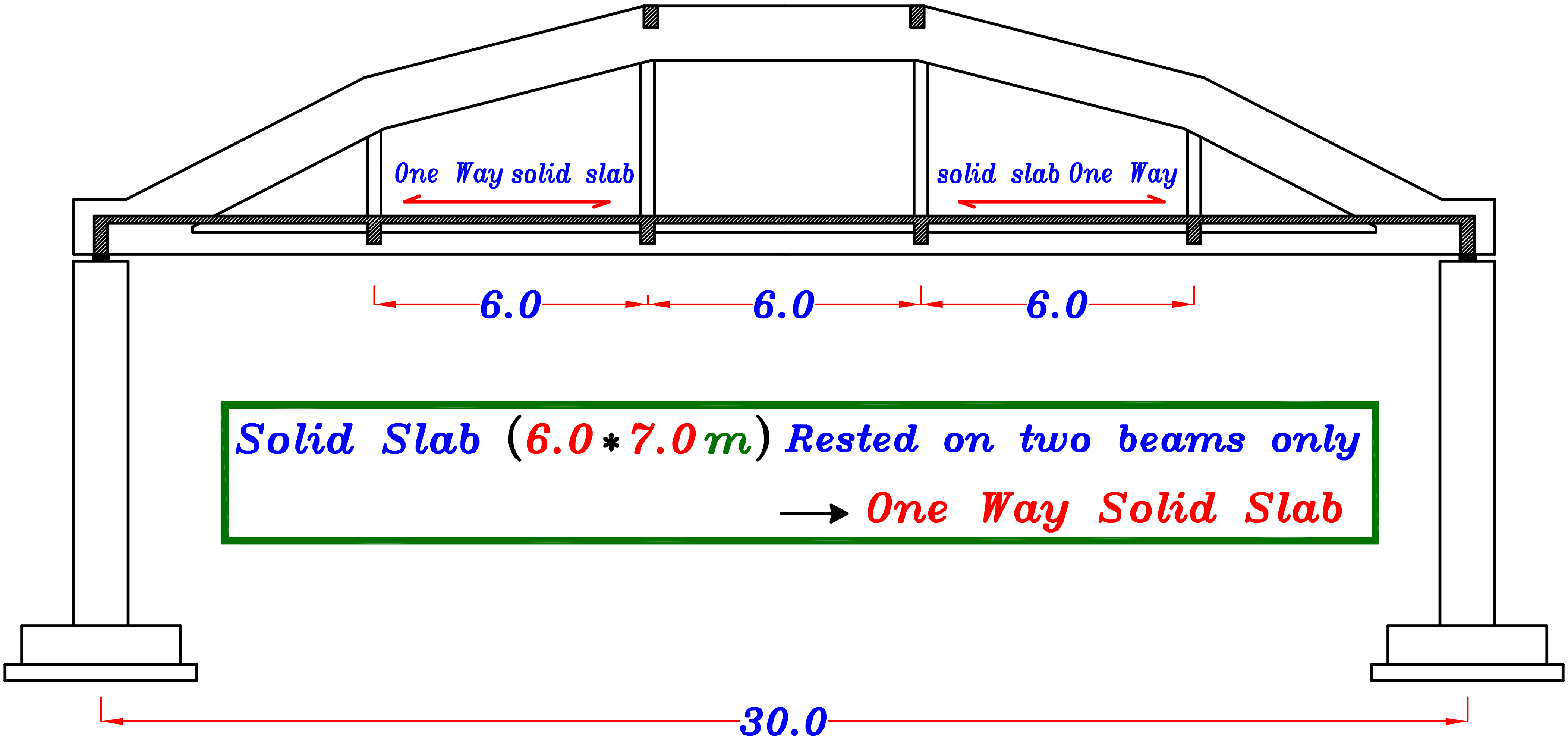
٢ـ أن نرفع البلاطه بحيث لا تُحمل على ال ***Arch Girder***



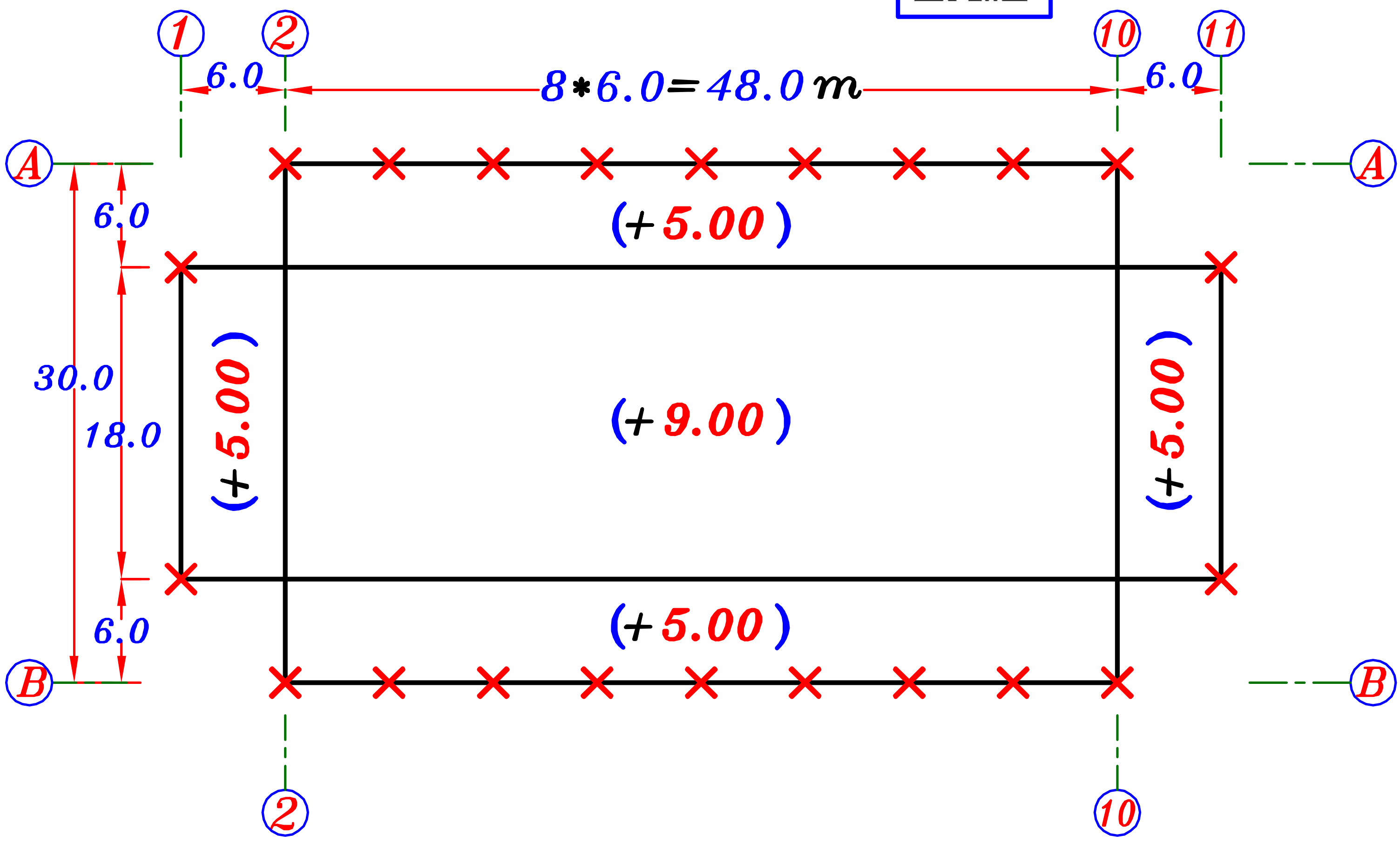
Solid Slab (3.0 * 7.0 m) → One Way Solid Slab

١ – أن نزيد من عدد الـ **segments** لـ **Arch Girder** بحيث تكون $a < \frac{S}{2}$

٢ـ أن نرفع البلاطه بحيث لا تُحمل على الـ Arch Girder



Example.



The Figure shows plan of the Factory area of an industrial Facility.

*The main Factory area is (30.0 * 48.0 m) with no interior columns allowed.*

Levels of the main Factory are as shown in plan.

Columns are only allowed where marked × in plan.

$F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

$F.C. = 3.0\ kN\backslash m^2$, $L.L. = 1.0\ kN\backslash m^2$

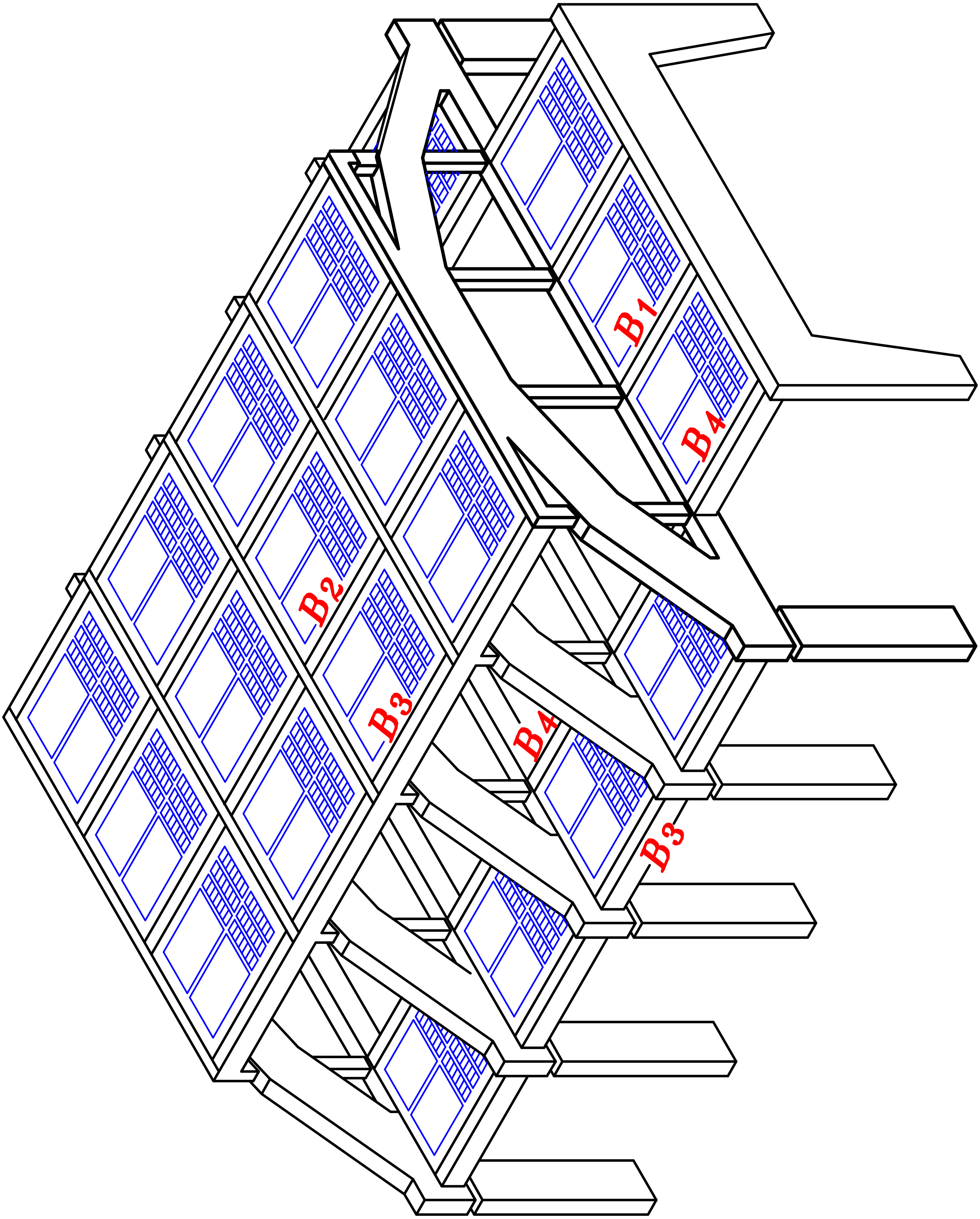
It is required :-

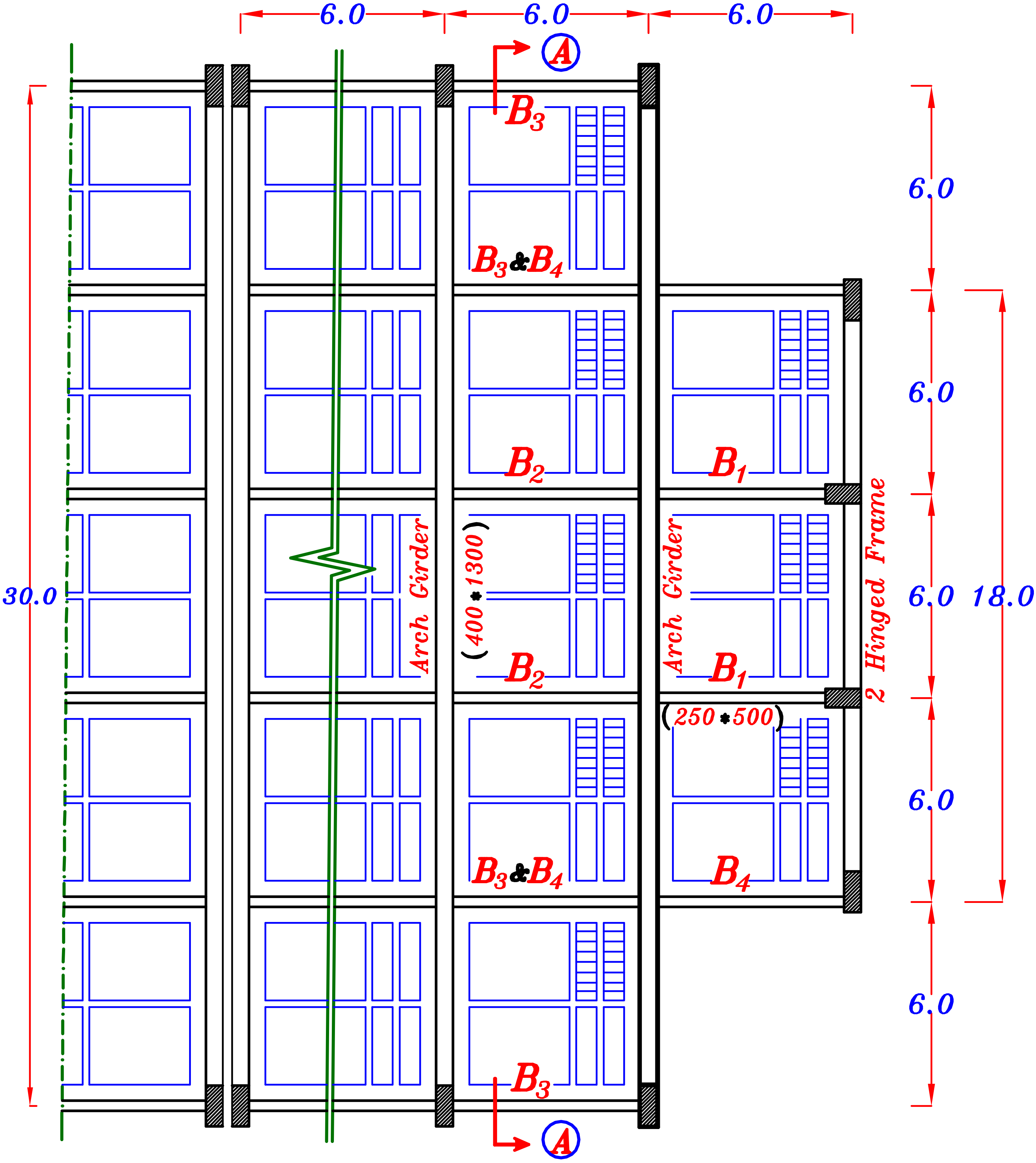
1- Without any calculation but with reasonably assumed concrete dimensions, Draw to scale 1:50 an elevation to the main supporting element at axis 10–10 and part plan.

2- Design all slabs and draw their details of reinforcement in plan.

3- Design the main supporting element of axis 10–10 of the Factory area.

4- Draw to a scale 1:50 in elevation and cross section details of reinforcement of the main supporting element.


B1
B4
B2
B3
B4
B3

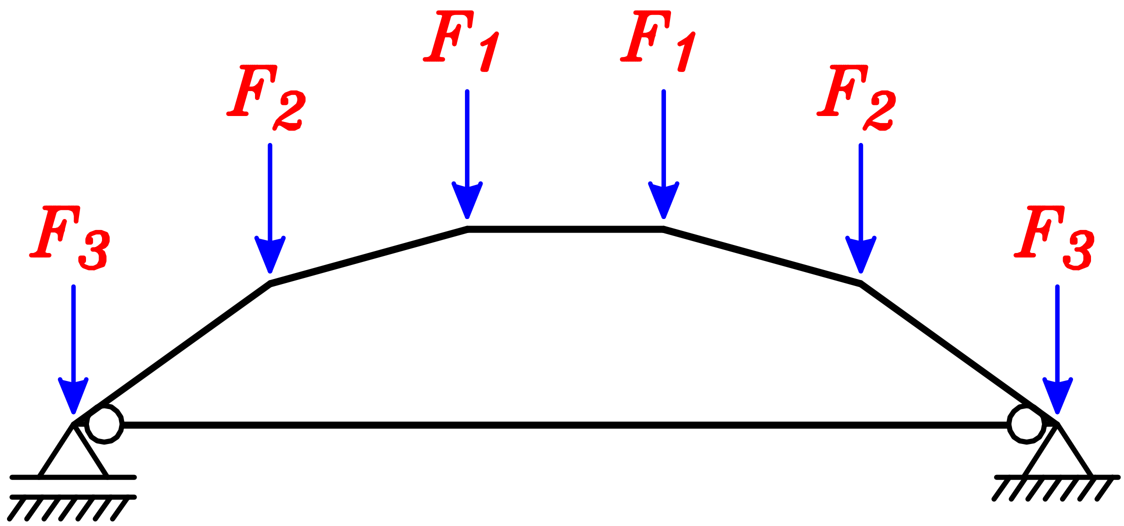


6.0
6.0
6.0
A
B3
B3&B4
B2
B1
Arch Girder
(400*1300)
B2
Arch Girder
B1
2 Hinged Frame
(250*500)
B3&B4
B4
B3
A
30.0
6.0
6.0
6.0
6.0
6.0
18.0

PLAN

Drawing Arch Girder at axis 10–10.

عند رسم ال Arch Girder عند axis 10–10 الاحمال عليه غير متساويه لذا عند تحديد ارتفاعاته لن ينفع أن نضع أحمال 1.0 kN عند كل Joint لذا يجب أن نرسم ال moment الحقيقى أولا ثم نحدد الارتفاعات على أساس أن نفس النسب بين الارتفاعات هى نفس النسب بين العزوم الحقيقيه



Loads on the Arch Girder at axis 10–10

Design the Slab.

For H.B. slab $t = \frac{6000}{25} = 240\ mm = 250\ mm$

$\boxed{t = 250\ mm}$ $\boxed{t_s = 50\ mm}$ $\boxed{h = 200\ mm}$

$$\boxed{\begin{aligned} w_{rib} &= \left[1.4\,(t_s \gamma_c + F.C.) + 1.6\,(L.L.)\right](S * 1.0) \\ &+ 1.4\,(b\,h * 1.0\,m * \gamma_c) + 1.4 * (\text{وزن ال Block})\left(\frac{1.0}{a}\right) \end{aligned}}$$

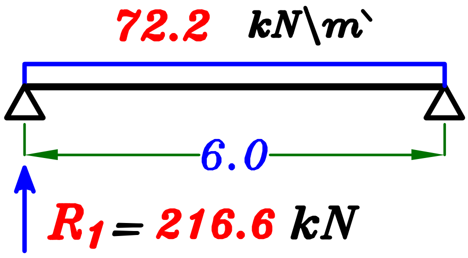
$$\therefore w_{rib} = \left[1.4\,(0.05 * 25 + 3.0) + 1.6\,(1.0)\right](0.50 * 1.0)$$
$$+ 1.4\,(0.10 * 0.20 * 1.0 * 25) + 1.4\left(\frac{160}{1000}\right)\left(\frac{1.0}{0.2}\right) = 5.60$$

$(kN \backslash (1.0 * S\ m^2))$

Loads Form Beams.

assume o.w.(beam) $= 5.0$ kN\m` (U.L.)

B_1



$$w = o.w. + \left(\frac{w_{rib}}{s}\right) L_s$$

$$w = 5.0 + \left(\frac{5.60}{0.5}\right)(6.0) = 72.2 \ kN\backslash m`$$

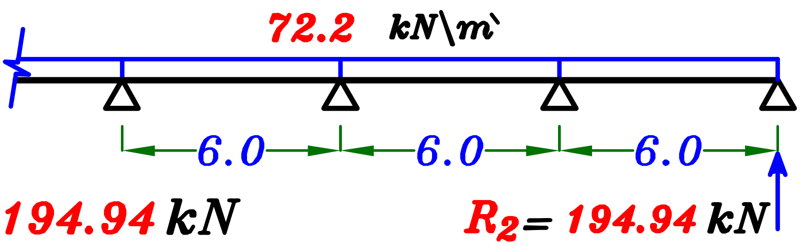
$$R_1 = w\frac{L}{2} = 72.2 * \frac{6.0}{2} = 216.6 \ kN$$

B_2

$$w = o.w. + \left(\frac{w_{rib}}{s}\right) L_s$$

$$w = 5.0 + \left(\frac{5.60}{0.5}\right)(6.0) = 72.2 \ kN\backslash m`$$

لان المطلوب تصميم Arch Girder عند axis ②
لذلك سنحتاج أول reaction للكمره B_2



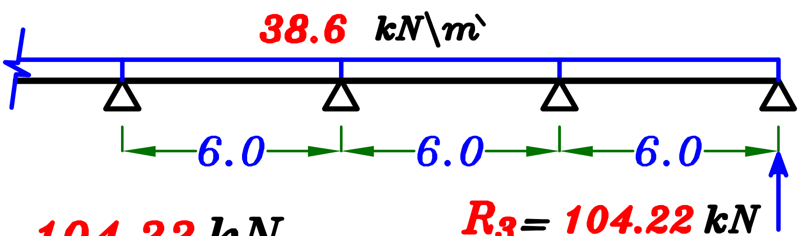
$$R_2 = 0.45 * w * L$$

$$R_2 = 0.45 * 72.2 * 6.0 = 194.94 \ kN$$

B_3

$$w = o.w. + \left(\frac{w_{rib}}{s}\right)\left(\frac{L_s}{2}\right) = 5.0 + \left(\frac{5.60}{0.5}\right)\left(\frac{6.0}{2}\right) = 38.6 \ kN\backslash m`$$

لان المطلوب تصميم Arch Girder عند axis ②
لذلك سنحتاج أول reaction للكمره B_3



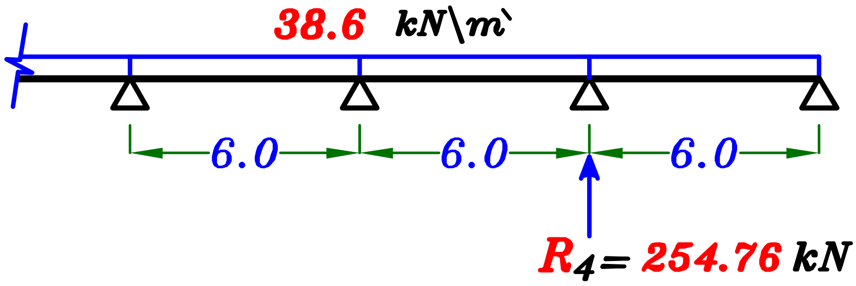
$$R_3 = 0.45 * w * L$$

$$R_3 = 0.45 * 38.6 * 6.0 = 104.22 \ kN$$

B_4

$$w = o.w. + \left(\frac{w_{rib}}{S}\right)\left(\frac{L_s}{2}\right) = 5.0 + \left(\frac{5.60}{0.5}\right)\left(\frac{6.0}{2}\right) = 38.6 \ kN\backslash m`$$

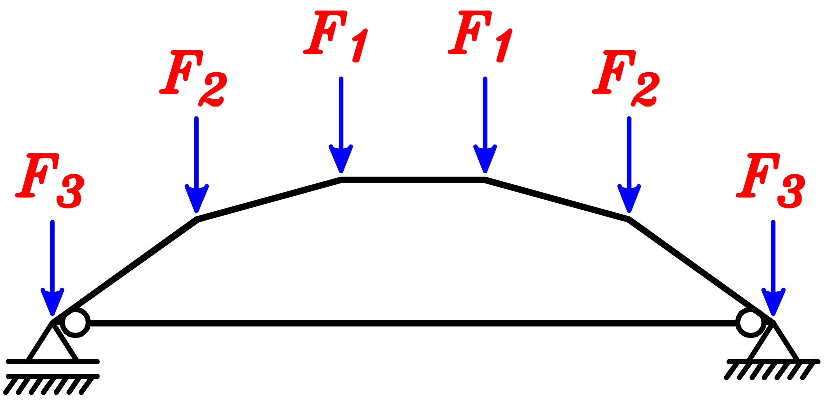
axis ② عند Arch Girder لان المطلوب تصميم
لذلك سنحتاج ثانى reaction للكمره B_4



$$R_4 = 1.10 * w * L$$

$$R_4 = 1.10 * 38.6 * 6.0 = 254.76 \ kN$$

Loads on Arch Girder.



Take $o.w.(Arch) = 17.5 \ kN\backslash m`$ (U.L.)

$$F_1 = R_1 + R_2 + o.w.(Arch) * a$$

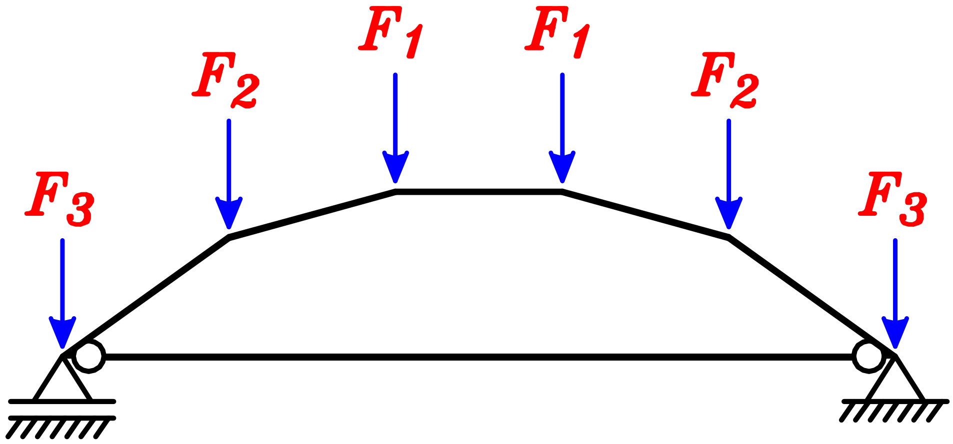
$$= 216.6 + 194.94 + 17.5\,(6.0) = 516.54 \ kN$$

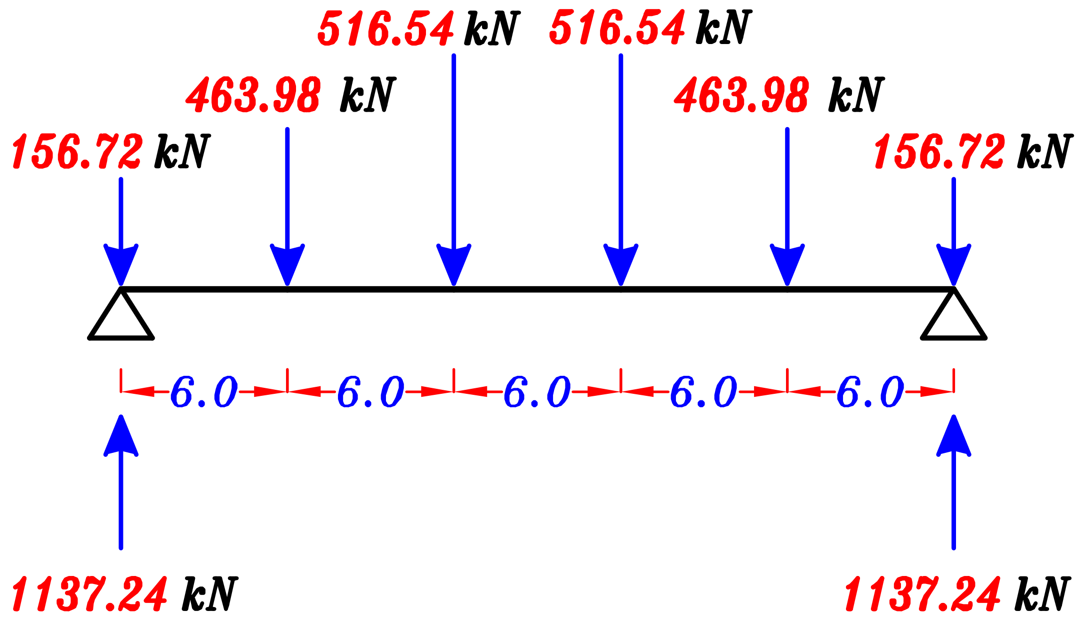
$$F_2 = R_3 + R_4 + o.w.(Arch) * a$$

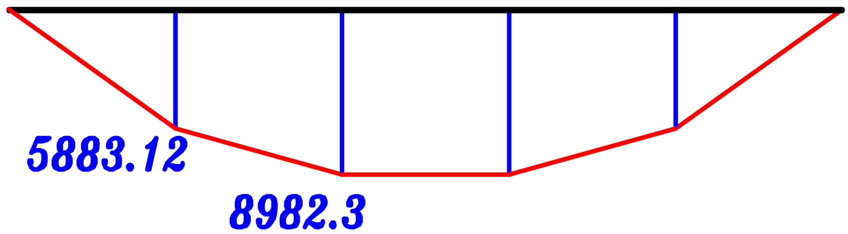
$$= 104.22 + 254.76 + 17.5\,(6.0) = 463.98 \ kN$$

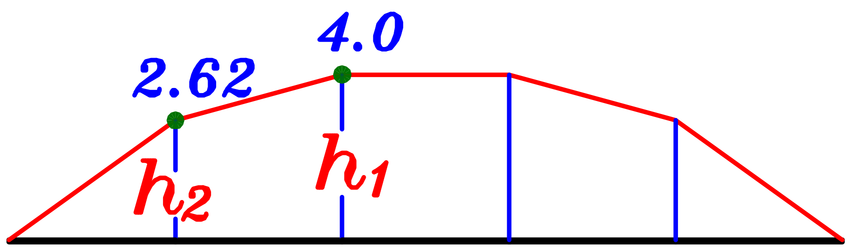
$$F_3 = R_3 + o.w.(Arch) * \frac{a}{2}$$

$$= 104.22 + 17.5\,(3.0) = 156.72 \ kN$$





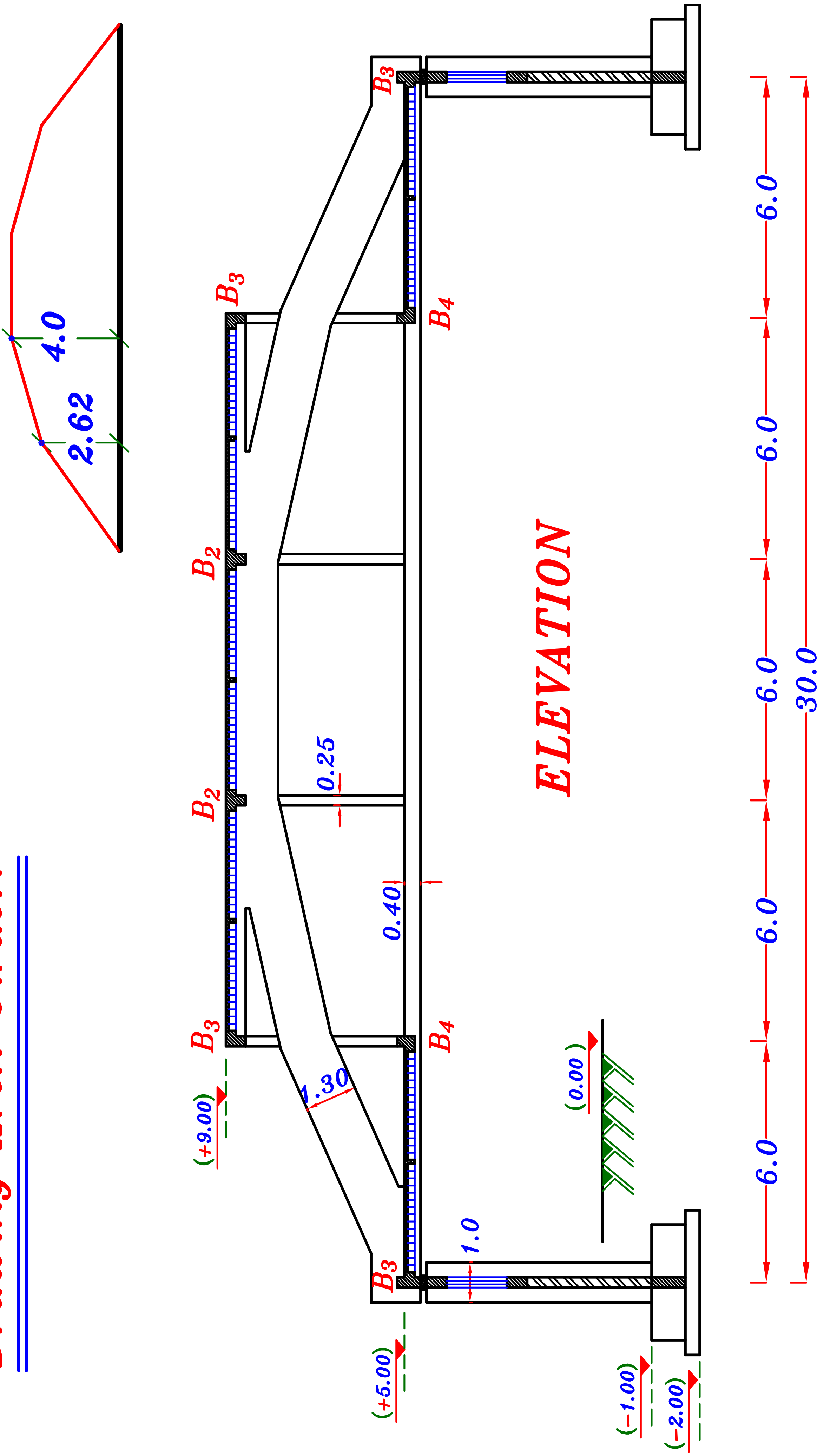




$$M_1 = 8982.3 \quad kN.m \longrightarrow h_1 = 4.0 \ m$$

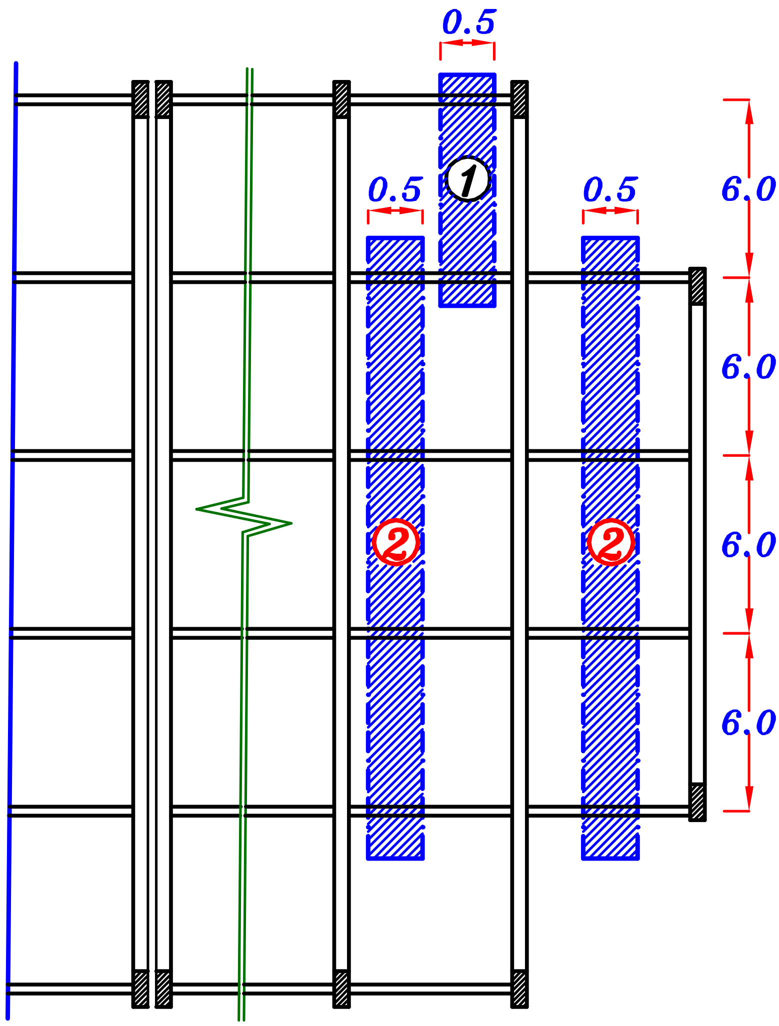
$$M_2 = 5883.12 \ kN.m \longrightarrow h_2 = 2.62 \ m$$

Drawing Arch Girder.

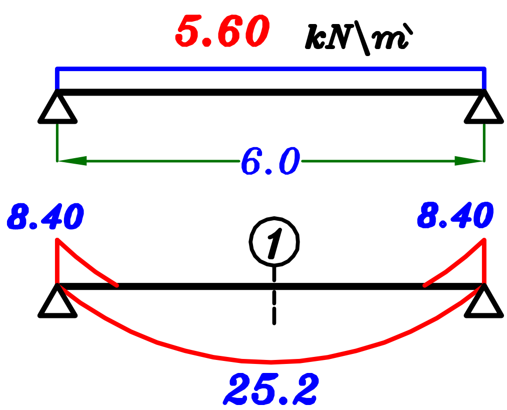


2 - Design the slabs.

$\therefore (w_{rib})_{U.L.} = 5.60 \; (kN \backslash (1.0 * 0.5\, m^2))$ as calculated before



Strip 1



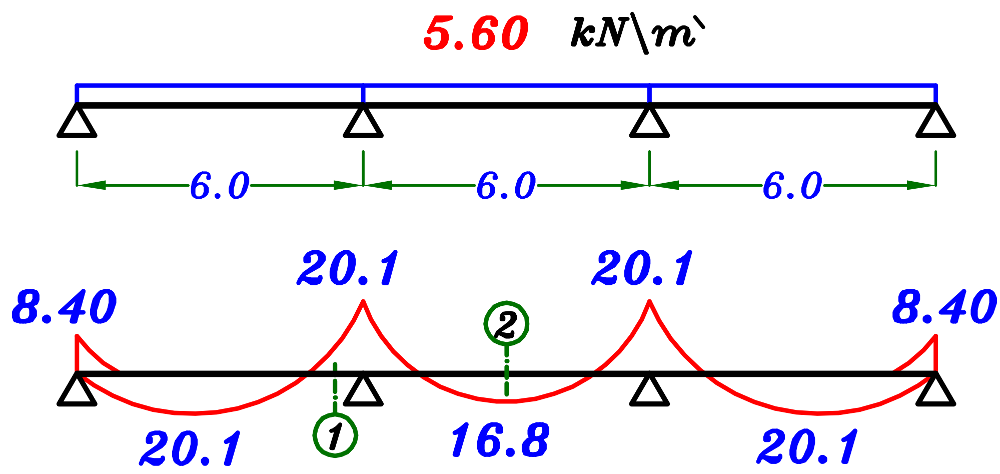
$M = 25.2$ kN.m\rib $\qquad d = t - 30\,mm = 250 - 30 = 220$ mm

$$d = C_1 \sqrt{\frac{M\ (kN.m\backslash rib)}{F_{cu}\ B}}$$

$$\therefore 220 = C_1 \sqrt{\frac{25.2 * 10^6}{25 * 500}} \longrightarrow C_1 = 4.89 \longrightarrow J = 0.826$$

$$A_s = \frac{M}{J F_y d} = \frac{25.2 * 10^6}{0.826 * 360 * 220} = 385 \; mm^2 \backslash rib \quad \boxed{2 \phi 16 \backslash rib}$$

Strip ②



Sec. ①

$M = 20.1$ kN.m\rib

$d = t - 30 \text{ mm} = 250 - 30 = 220 \text{ mm}$

$$\therefore 220 = C_1 \sqrt{\frac{20.1 * 10^6}{25 * 500}} \rightarrow C_1 = 5.48 \rightarrow J = 0.826$$

$$A_s = \frac{M}{J\,F\,d} = \frac{20.1 * 10^6}{0.826 * 360 * 220} = 307 \text{ mm}^2\text{\rib}$$

$\boxed{2 \phi 16 \text{\rib}}$

Sec. ②

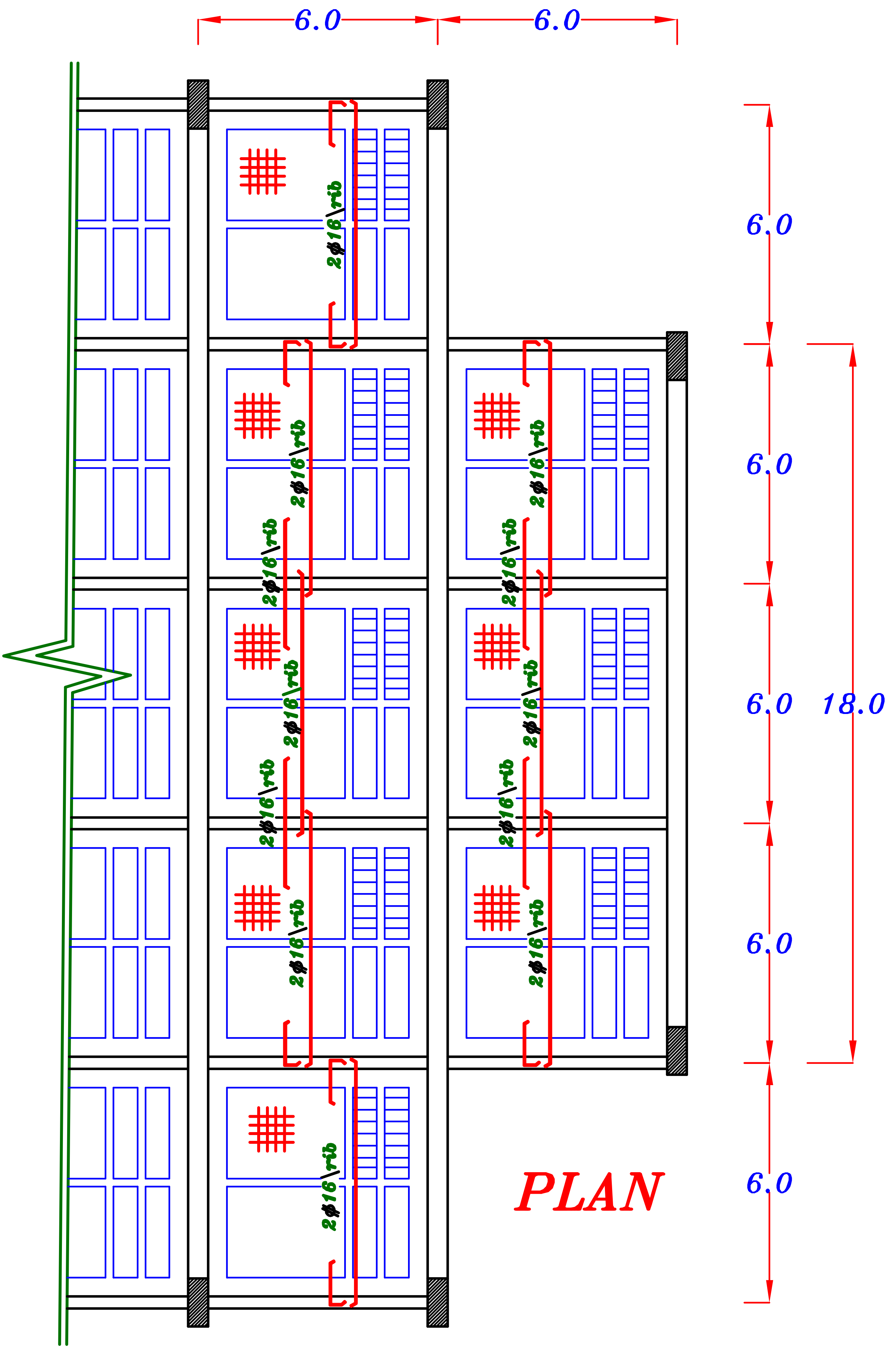
$M = 16.8$ kN.m\rib

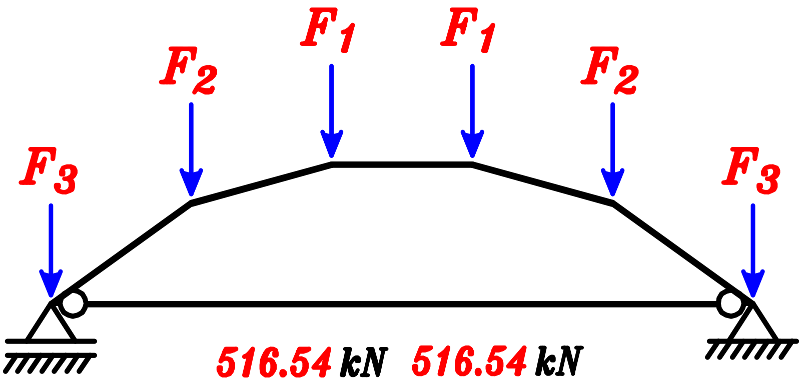
$d = t - 30 \text{ mm} = 250 - 30 = 220 \text{ mm}$

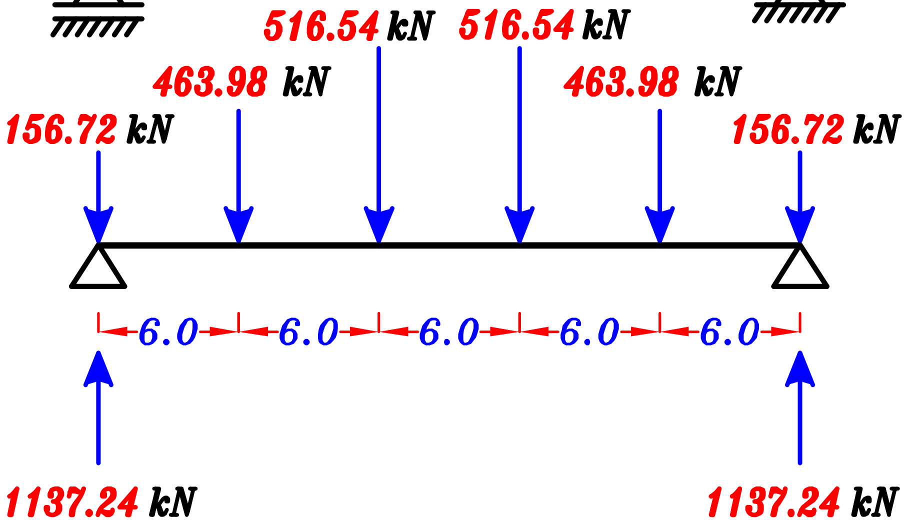
$$\therefore 220 = C_1 \sqrt{\frac{16.8 * 10^6}{25 * 500}} \rightarrow C_1 = 6.0 \rightarrow J = 0.826$$

$$A_s = \frac{M}{J\,F\,d} = \frac{16.8 * 10^6}{0.826 * 360 * 220} = 256 \text{ mm}^2\text{\rib}$$

$\boxed{2 \phi 16 \text{\rib}}$


6.0
6.0
6.0
6.0
6.0
18.0
6.0
6.0
2φ16\rib
PLAN

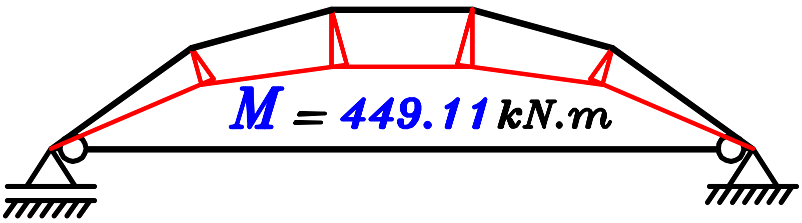




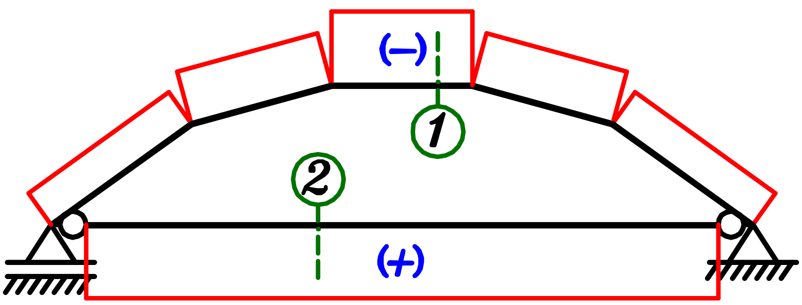
$$M_o = 8982.3 \text{ kN.m}$$

$$P = T = 0.95\,\frac{M_o}{h} = 0.95 * \frac{8982.3}{4.0} = 2133.3 \text{ kN}$$

$$M = 0.05\, M_o = 0.05\,(8982.3) = 449.11 \text{ kN.m}$$



$$P = 2133.3 \text{ kN}$$



$$T = 2133.3 \text{ kN}$$

* Design of Arch Girder.

Sec. ① $b = 0.35\ m$, $t = 1.30\ m$

$P = 2133.3\ kN$, $M = 449.11\ kN.m$

$$e = \frac{M}{P} = \frac{449.11}{2133.3} = 0.21\ m$$

$$\therefore \frac{e}{t} = \frac{0.21}{1.30} = 0.161\ m < 0.5 \xrightarrow{use} I.D.$$

$$\zeta = \frac{1.3 - 0.2}{1.3} = 0.84 = 0.80 \xrightarrow{use} \text{ECCS Design Aids Page } 4-21$$

$$\left.\begin{array}{l} \dfrac{P_U}{F_{cu}\, b\, t} = \dfrac{2133.3 * 10^3}{25 * 400 * 1300} = 0.164 \\[2ex] \dfrac{M_U}{F_{cu}\, b\, t^2} = \dfrac{449.11 * 10^6}{25 * 400 * 1300^2} = 0.0265 \end{array}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 25 * 10^{-4} = 2.5 * 10^{-3}$$

$$A_s = A_s` = \mu * b * t = 2.5 * 10^{-3} * 400 * 1300 = 1300\ mm^2$$

$$A_{s\,Total} = A_s + A_s` = 2 * 1300 = 2600\ mm^2$$

– Check $A_{s\,min.} = \dfrac{0.8}{100} * b * t = \dfrac{0.8}{100} * 400 * 1300 = 4160 > A_{s\,Total}$

Take $A_s = A_s` = \dfrac{A_{s\,min}}{2} = \dfrac{4160}{2} = 2080\ mm^2$ $6 \phi 22$

$$n = \frac{b - 25}{\phi + 25} = \frac{400 - 25}{22 + 25} = 7.97 = 7.0 \text{ bars}$$

* Design of Tie.

Sec. ② (400 * 400) $T = 2133.3$ kN

$M =$ (due to o.w. only) Can be neglected.

$$A_s = \frac{T}{F_y \backslash \gamma_s} = \frac{2133.3 * 10^3}{360 \backslash 1.15} = 6814.7\ mm^2$$ 14 ϕ 25

* Design of the hangers. (250 * 250)

Take o.w.(hanger) = 3.50 kN (U.L.)

$T =$ o.w.(hanger) $+ R_1 = 3.50 + 216.6 = 220.1$ kN

$$A_s = \frac{T}{F_y \backslash \gamma_s} = \frac{220.1 * 10^3}{360 \backslash 1.15} = 703.1\ mm^2$$ 4 ϕ 16

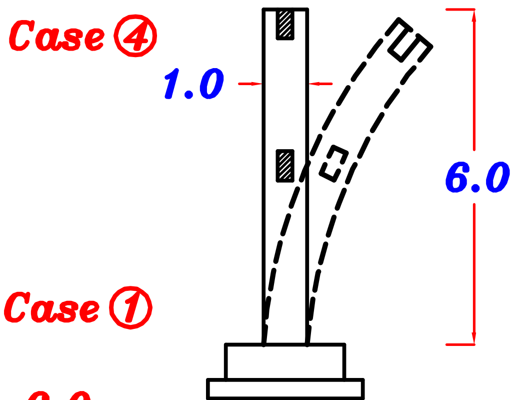
---

* Design of Columns. (neglect the effect of wind)

$$P = \frac{\Sigma F}{2} = 1137.24 \text{ kN}$$

Check Buckling.

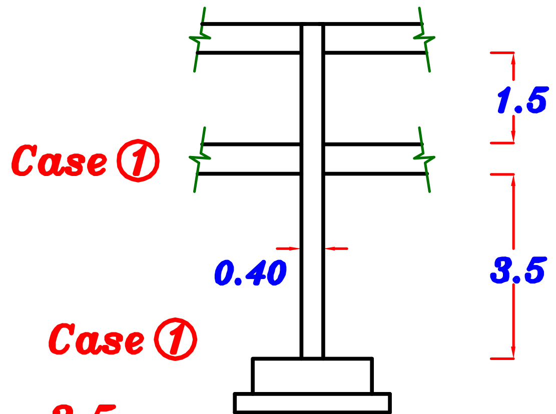
① In Plane.



$H_o = 6.0$ m

$$\lambda_b = \frac{K * H_o}{t} = \frac{2.2 * 6.0}{1.0} = 13.2 > 10$$

② Out of Plane.



$H_o = 3.5$ m

$$\lambda_b = \frac{K * H_o}{t} = \frac{1.2 * 3.5}{0.40} = 10.5$$

$$\delta = \frac{(\lambda_b)^2 * t}{2000} = \frac{13.2^2 * 1.0}{2000} = 0.087 \ m$$

$$M_{add.} = P * \delta = 1137.24 * 0.087 = 98.94 \ kN.m$$

$$e = \frac{M}{P} = \frac{98.94}{1137.24} = 0.087 \ m \quad \therefore \frac{e}{t} = \frac{0.087}{1.0} = 0.087 < 0.5 \xrightarrow{use} I.D.$$

$$\zeta = \frac{1.0 - 0.1}{1.0} = 0.9 \xrightarrow{use} \text{ECCS Design Aids Page } 4-23$$

$$\left.\begin{array}{l} \dfrac{P_U}{F_{cu}\, b\, t} = \dfrac{1137.24 * 10^3}{25 * 400 * 1000} = 0.113 \\ \dfrac{M_U}{F_{cu}\, b\, t^2} = \dfrac{98.94 * 10^6}{25 * 400 * 1000^2} = 0.0099 \end{array}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 25 * 10^{-4} = 2.5 * 10^{-3}$$

$$A_s = A_{s`} = \mu * b * t = 2.5 * 10^{-3} * 400 * 1000 = 1000 \ mm^2$$

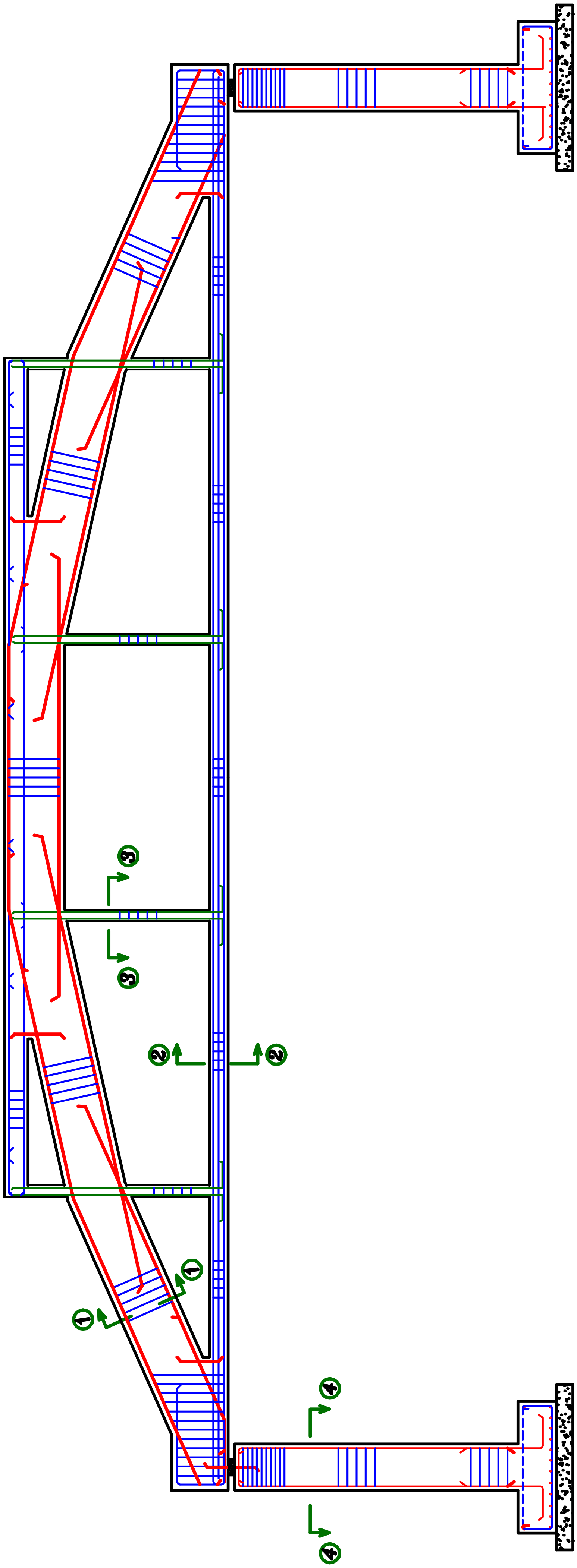
$$A_{s\,Total} = A_s + A_{s`} = 2 * 1000 = 2000 \ mm^2$$

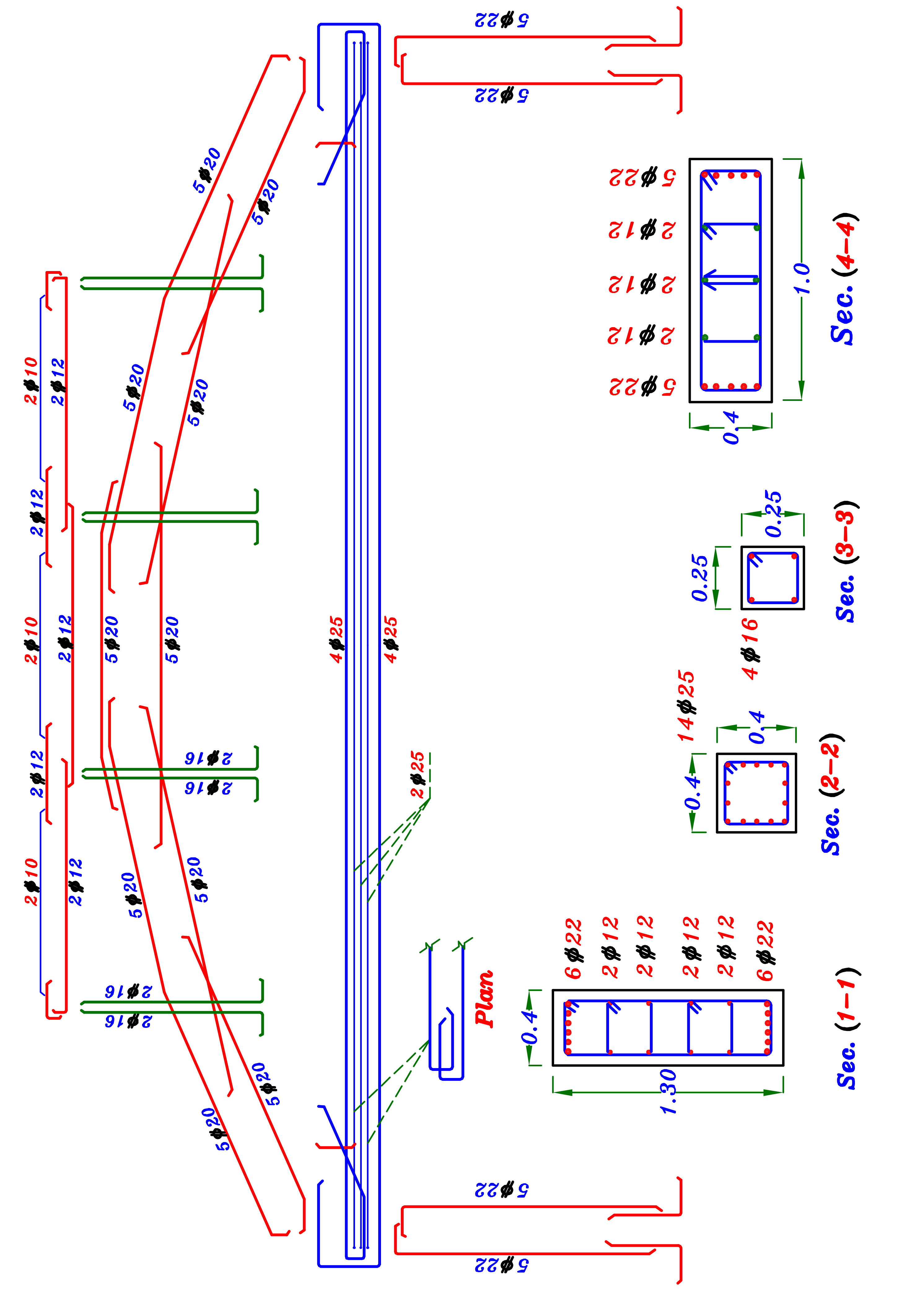
$$A_{s_{min}} = \frac{0.25 + 0.052 \lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052 (13.2)}{100} * 400 * 1000 = 3745 \ mm^2 > A_{s\,total}$$

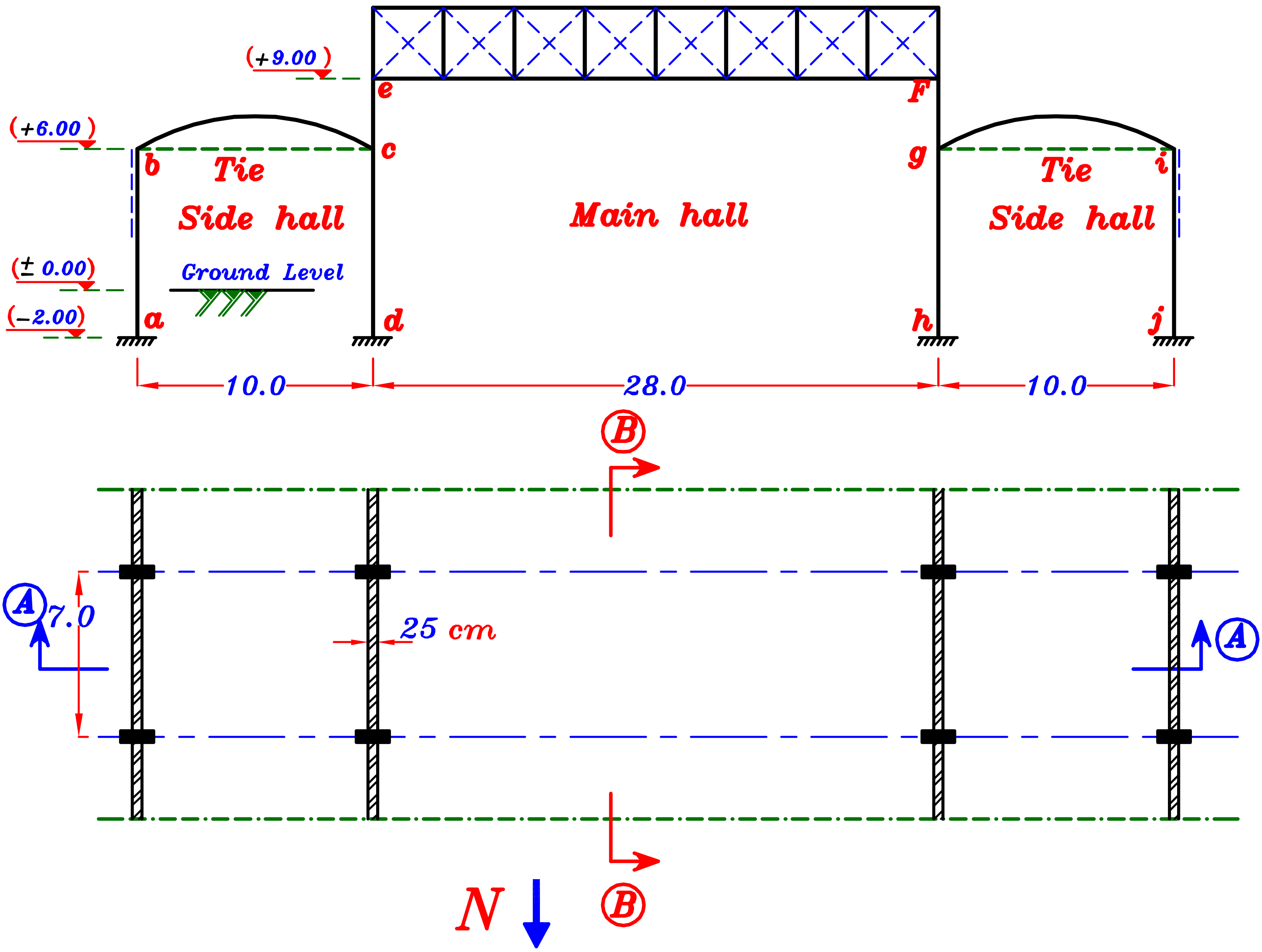
$$A_s = A_{s`} = \frac{A_{s\,min}}{2} = 1873 \ mm^2$$

5 ϕ 22




Plan
Sec. (1–1)
Sec. (2–2)
Sec. (3–3)
Sec. (4–4)
5φ22
2φ12
2φ10
2φ16
5φ20
4φ25
2φ25
6φ22
14φ25
4φ16
1.30
0.4
0.25
1.0

Example.



The Fig. Shows the general layout of Sec. (A−A) & Plan of an Industrial building covering an area (48∗42 m) The building consists of a main hall (28∗42 m) & 2 Side halls (10∗42 m)

The roof of the main hall is of the Saw Tooth type Facing the North

The side halls are covered with an arched slab with a tie

The Columns & the Ties of the side halls are Placed at spacing 7.0 m at the Longitudinal direction.

The Foundation Level is 2.0 m below the ground level.

Brick walls are 25 cm thickness are placed along the perimeter between the columns in the longitudinal direction.

Design Data :

* $F_{cu} = 25$ N\mm²
* $F_y = 360$ N\mm²
* Total loads (D.L.+L.L.) of the saw tooth roof are 8.0 kN\m² H.P.
* Covering (F.C.+L.L.) of the arch slab roof are 1.0 kN\m² H.P.
* Weight of 25 cm. thick brick wall is 4.0 kN\m²

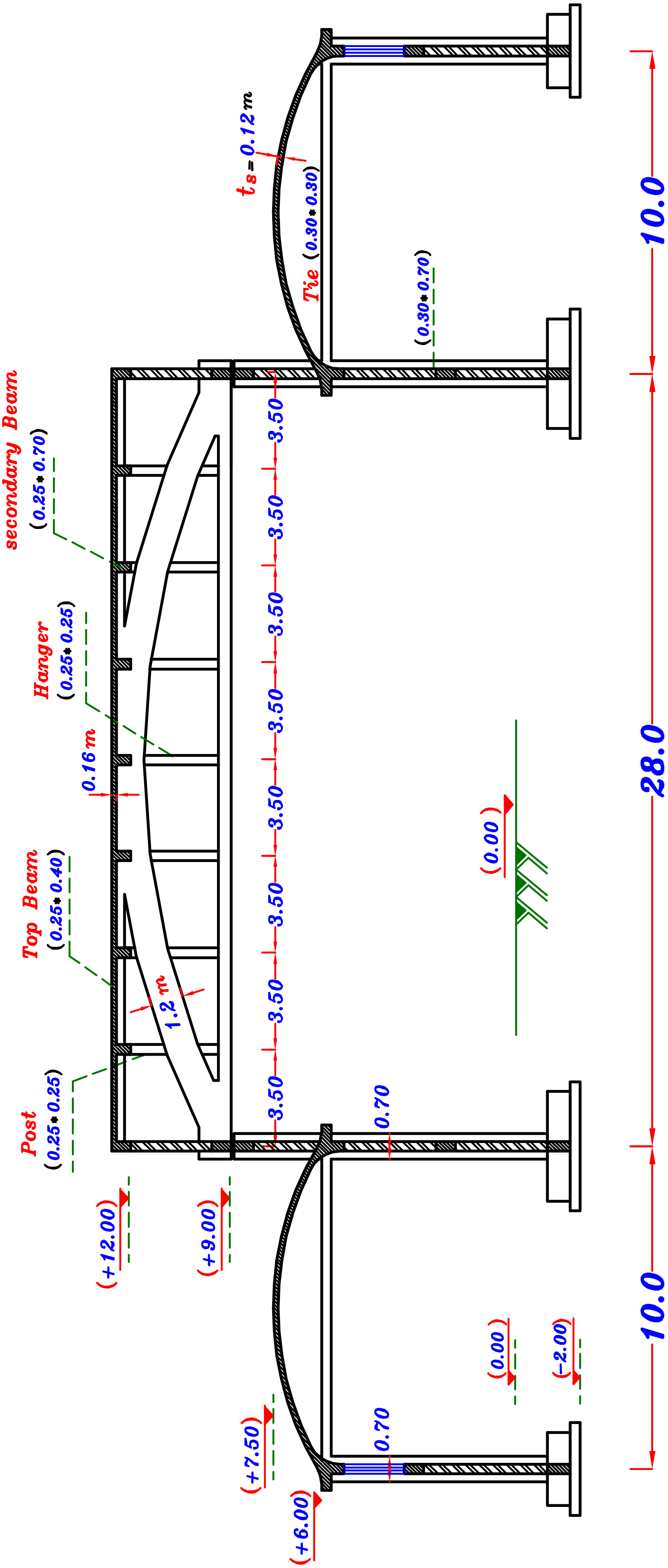
Required :

1- Without any calculations, but with reasonably assumed concrete Dim. draw a sectional elevation For the main system. scale (1:100)

2- Design the arch slab roof (arched slab and all it's supporting elements)
& show clearly the details of reinforcements For the arched slab and all it's supporting elements.

3- Design the saw tooth slab and it's elements.
and show the details of reinforcement on a plan to scale 1:50

4- For the main hall, design an intermediate system and it's elements.
Wind load are to be neglected .
& Draw to scale (1:50) a sectional elevation and to scale (1:25) cross sections for the main system showing all reinforcement details.

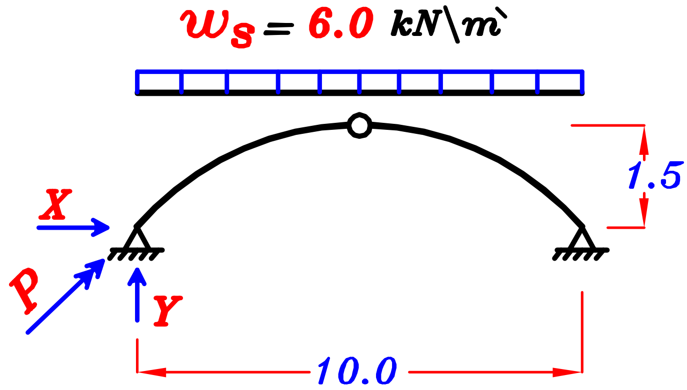


Sec. Elevation A-A

② Design the Arch Slab.

Take $t_s = 120$ mm

$\because$ F.C. + L.L. $= 1.0\ kN\backslash m^2$ H.P.



$$(w_s)_{U.L.} = 1.5\,(t_s\,\gamma_c + F.C. + L.L.)$$

$$(w_s)_{U.L.} = 1.5\,(0.12 * 25 + 1.0)$$

$$= 6.0\ kN\backslash m^2\ \ (H.P.)$$

To Get N.F.

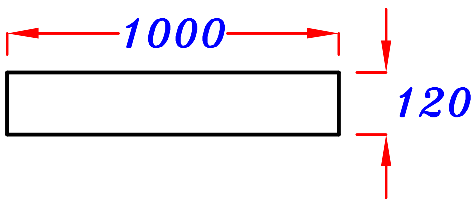
$$Y = \frac{w\,L}{2} = \frac{6.0 * 10}{2} = 30.0\ kN\backslash m$$

$$X = \frac{w\,L^2}{8\,h} = \frac{6.0 * 10^2}{8 * 1.5} = 50.0\ kN\backslash m$$

$$P = \sqrt{X^2 + Y^2} = \sqrt{50^2 + 30^2} = 58.3\ kN$$

* Design the Arch Slab.

Neglect B.M. & Design on N.F. only.



$\therefore$ Designed as a Column.

$$\therefore\ P_{U.L.} = 0.35\,A_c\,F_{cu} + 0.67\,A_s\,F_y$$

Take $A_c = 120 * 1000 = 120000\ mm^2$

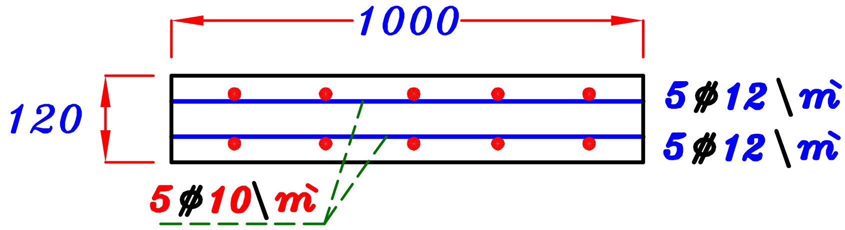
$$\therefore\ 58.3 * 10^3 = 0.35\,(120000)(25) + 0.67\,A_s\,(360)$$

$\therefore\ A_s = -\ 4111.5\ mm^2 = (-\ Ve)$ Value

$$\therefore\ \text{Take}\ A_s = A_{s\,min.} = \frac{0.8}{100} * b * t$$

$$\therefore\ A_s = \frac{0.8}{100} * 120 * 1000 = 960\ mm^2 = A_{s\,total}$$

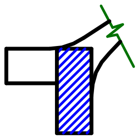
$$\therefore\ \text{Upper Steel \& Lower Steel} = \frac{A_{s\,total}}{2} = \frac{960}{2} = 480\ mm^2$$



5φ12\m'

Design of End Beam.

VL. Beam.

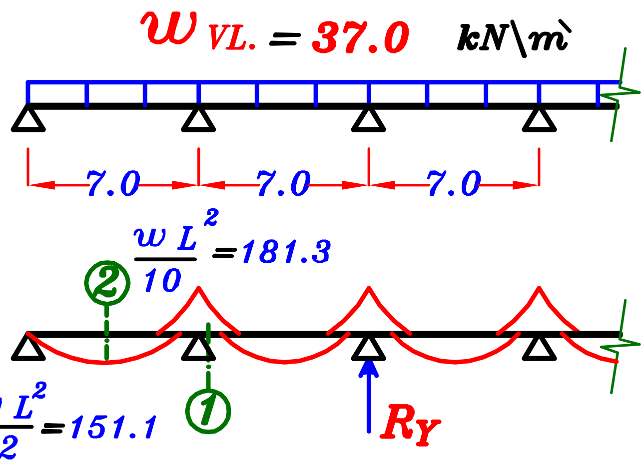


Take $O.W._{(beam)}$ (VL.+HL.) $\approx$ 7.0 kN\m` (U.L.)

$$w_{VL.} = O.W._{(beam)} + Y = 7.0 + 30.0 = 37.0 \text{ kN\m`}$$

$$R_Y = w_{VL.} * S = 259.0 \text{ kN}$$

Design all Sections as R–Sec.



Sec. ①

$M_{U.L.} = 181.3$ kN.m R–Sec.

- Take $C_1 = 3.50 \longrightarrow J = 0.78$

- Get $d = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, b}} = 3.50 \sqrt{\dfrac{181.3 * 10^6}{25 * 250}} = 596$ mm

- Take $\boxed{d = 600 \text{ mm}}$, $\boxed{t = 650 \text{ mm}}$

- Get $A_s = \dfrac{M_{U.L.}}{J\, F_y\, d} = \dfrac{181.3 * 10^6}{0.78 * 360 * 596} = 1083.3 \text{ mm}^2$

Check $A_{s\,min.}$

$$A_{s_{req.}} = 1083.3 \text{ mm}^2$$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 600 = 468.75 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 1083.3 \text{ mm}^2$ 6 ϕ16

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{16 + 25} = 5.48 = 5.0 \text{ bars}$$

Sec. 2 $M_{U.L.} = 151.1$ kN.m

$$600 = C_1 \sqrt{\frac{151.1 * 10^6}{25 * 250}} \longrightarrow C_1 = 3.85 \longrightarrow J = 0.797$$

$$A_s = \frac{151.1 * 10^6}{0.797 * 360 * 600} = 877.7 \ mm^2$$

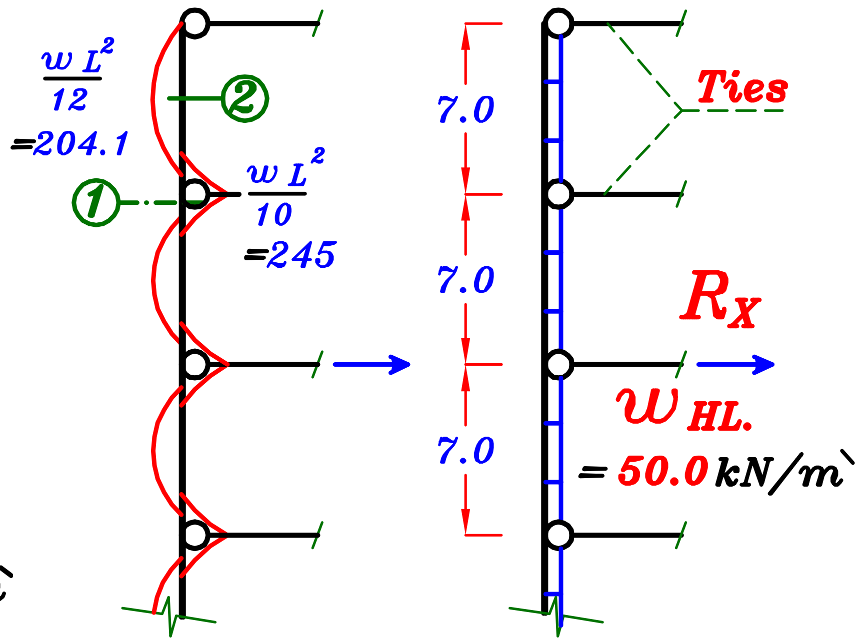
Check $A_{s\,min.}$ $\quad A_{s_{req.}} = 877.7 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 600 = 468.75 \ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 877.7 \ mm^2$ $\quad$ 5 ϕ16

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\ 877.7$ $\quad$ 2 ϕ10

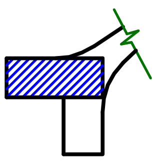
HL. Beam.



$w_{HL.} = X = 50.0$ kN\m`

$R_X = w_{HL.} * S = 350$ kN

Design all Sections as R-Sec.



Sec. ① $M_{U.L.} = 245.0$ kN.m R–Sec.

- Take $C_1 = 3.50 \longrightarrow J = 0.78$

- Get $d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} = 3.50 \sqrt{\frac{245.0 * 10^6}{25 * 250}} = 692$ mm

- Take $\boxed{d = 700\,mm}$, $\boxed{t = 750\,mm}$

- Get $A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{245.0 * 10^6}{0.78 * 360 * 692} = 1260.8\ mm^2$

Check $A_{s\,min.}$ $\quad A_{s_{req.}} = 1260.8\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 700 = 546.87\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1260.8\ mm^2$ $\quad$ 7 ϕ16

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{16 + 25} = 5.48 = 5.0 \text{ bars}$$

Sec. ② $M_{U.L.} = 204.1$ kN.m

$$700 = C_1 \sqrt{\frac{204.1 * 10^6}{25 * 250}} \longrightarrow C_1 = 3.87 \longrightarrow J = 0.799$$

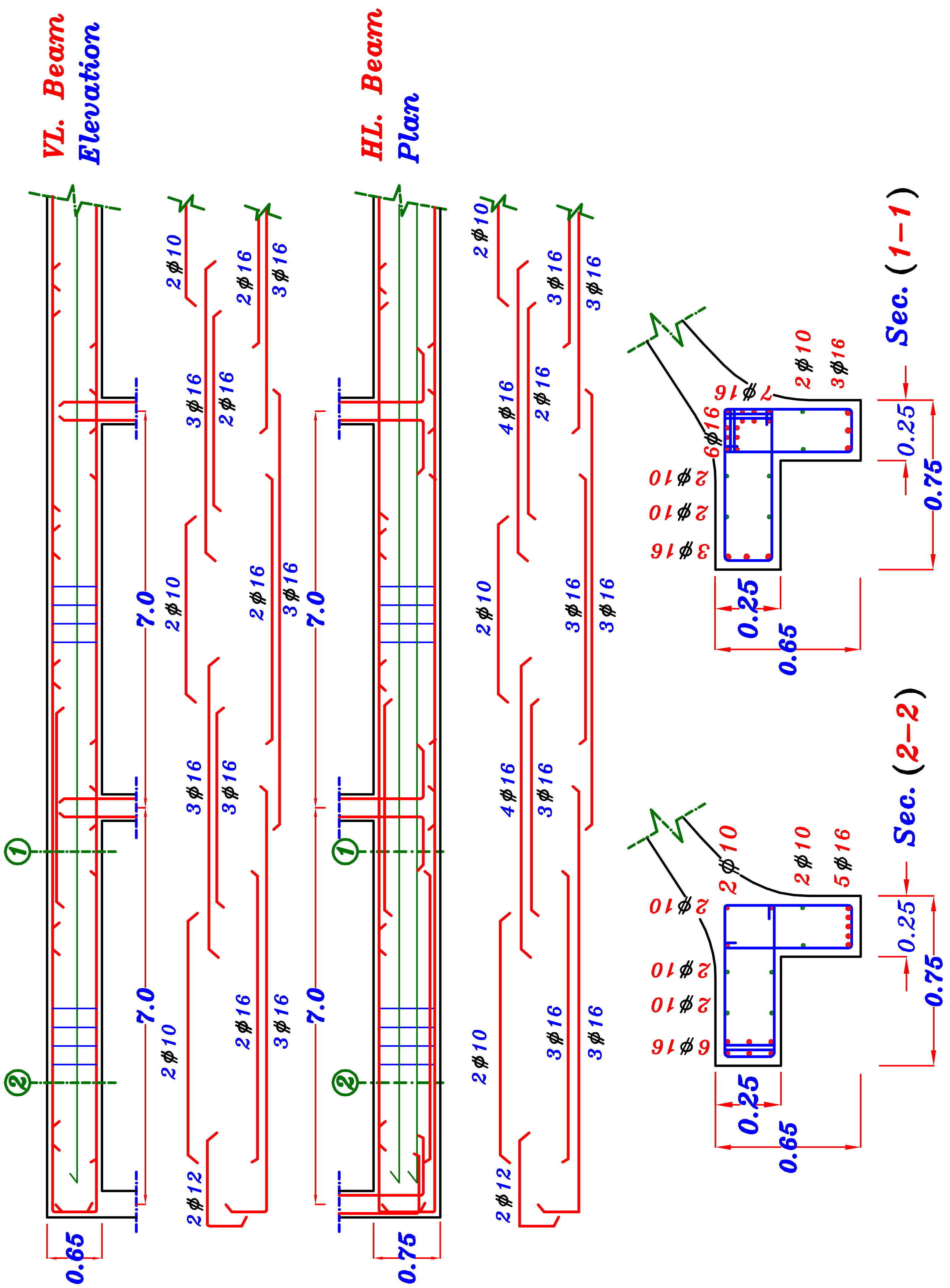
$$A_s = \frac{204.1 * 10^6}{0.799 * 360 * 700} = 1013.7\ mm^2$$

Check $A_{s\,min.}$ $\quad A_{s_{req.}} = 1013.7\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 700 = 546.87\ mm^2$$

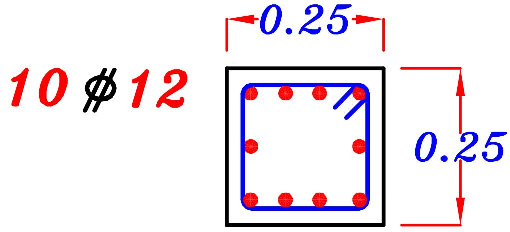
$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1013.7\ mm^2$ $\quad$ 6 ϕ16

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\ 1013.6$ $\quad$ 2 ϕ10


VL. Beam
Elevation
HL. Beam
Plan
Sec. (1-1)
Sec. (2-2)
7.0
0.65
0.75
0.25
2 ϕ 10
2 ϕ 12
2 ϕ 16
3 ϕ 16
4 ϕ 16
5 ϕ 16
6 ϕ 16
7 ϕ 16

* Design the Tie. (250 * 250)

Neglect o.w. of Tie.



$$T_{(Tie)} = R_X = 350 \text{ kN}$$

$$A_S = \frac{T}{F_y \backslash \delta_s} = \frac{350 * 10^3}{360 \backslash 1.15} = 1118 \text{ mm}^2 \qquad 10 \phi 12$$

Design the Column. (350 * 700)

w = Loads From Saw Tooth Slab\m² * S + O.W.(Arch Girder)

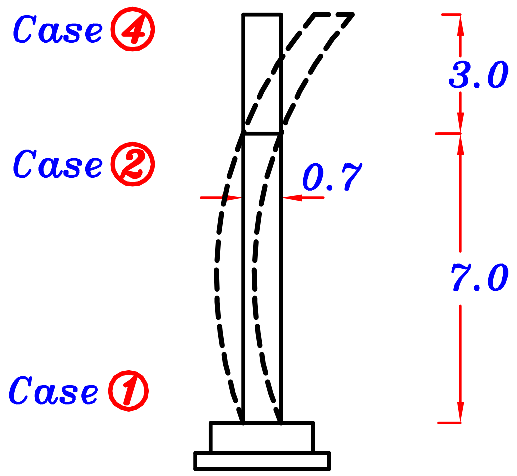
$$1.5 * 8.0 * 7.0 \text{ kN}\backslash\text{m} + 17.5 \text{ kN}\backslash\text{m} = 101.5 \text{ kN}\backslash\text{m}$$

$$R = w * \frac{L}{2} = 101.5 * \frac{28}{2} = 1421 \text{ kN}$$

$$P = R + R_Y = 1421 + 259.0 = 1680 \text{ kN}$$

Check Buckling. $P = 1680$ kN

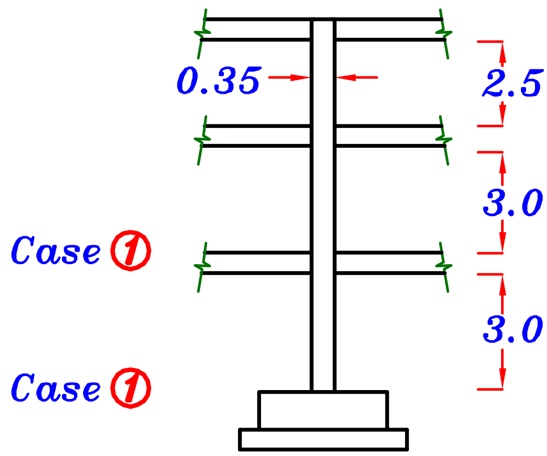
① In Plane.



$$H_o = 7.0 \text{ m}$$

$$\lambda_b = \frac{K * H_o}{t} = \frac{1.3 * 7.0}{0.7} = 13 > 10$$

② Out of Plane.



$$H_o = 3.0 \text{ m}$$

$$\lambda_b = \frac{K * H_o}{b} = \frac{1.2 * 3.0}{0.35} = 10.28$$

$$\delta = \frac{(\lambda_b)^2 * t}{2000} = \frac{13.0^2 * 0.70}{2000} = 0.059 \ m$$

$$M_{add.} = P * \delta = 1680 * 0.059 = 99.12 \quad kN.m$$

$$e = \frac{M}{P} = \frac{99.12}{1680} = 0.059 \ m \qquad \therefore \frac{e}{t} = \frac{0.059}{0.7} = 0.08 < 0.5 \xrightarrow{use} I.D.$$

$$\zeta = \frac{0.7-0.1}{0.7} = 0.85 = 0.80 \xrightarrow{use} \text{ECCS Design Aids Page } 4-24$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{1680 * 10^3}{25*350*700} = 0.27 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{99.12 * 10^6}{25*350*700^2} = 0.023 \end{aligned}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

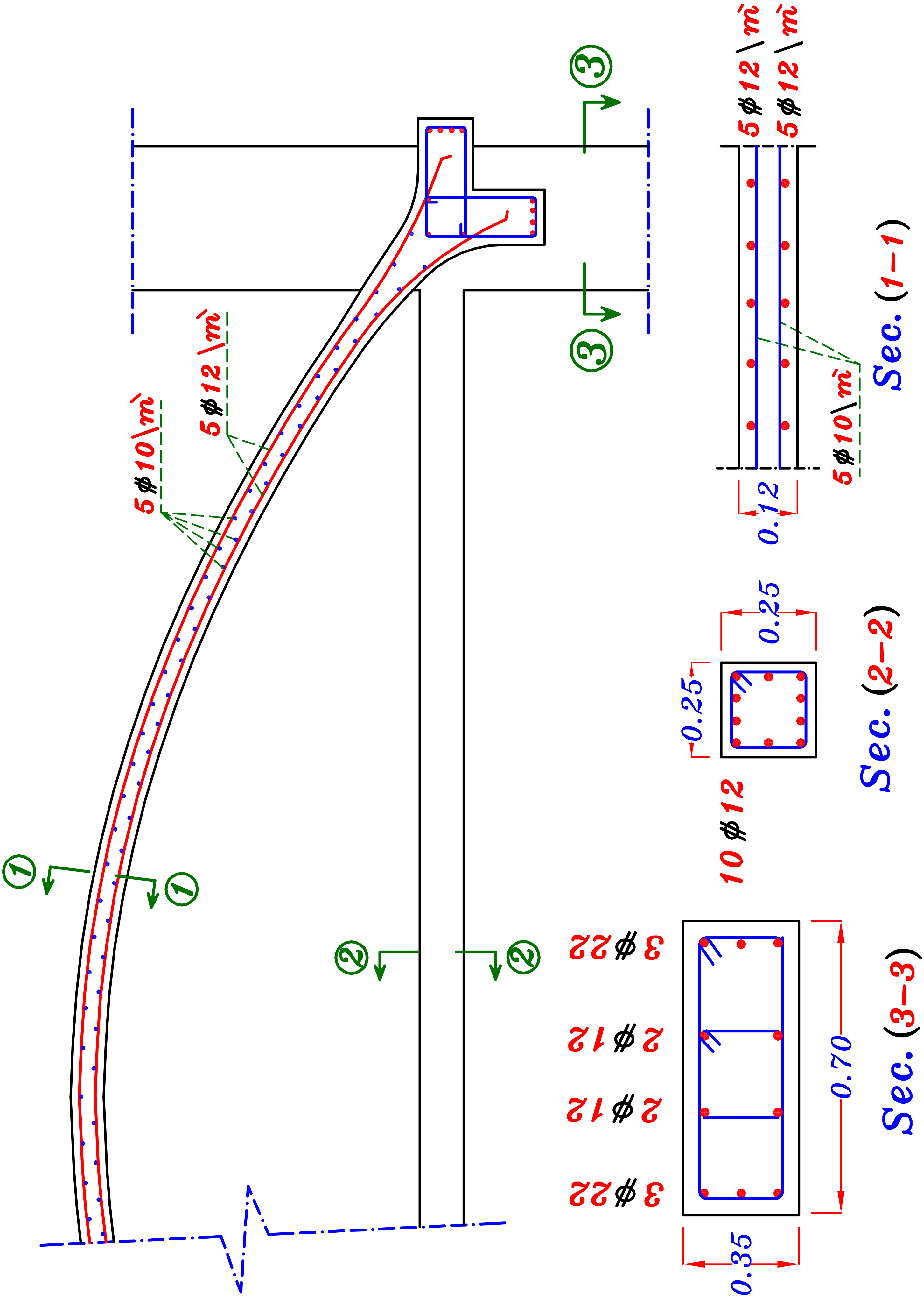
$$As = As` = \mu * b * t = \rho * F_{cu} * 10^{-4} * b * t = 1.0 * 25 * 10^{-4} * 350 * 700 = 612.5 \ mm^2$$

$$As_{total} = As + As` = 1225 \ mm^2$$

$$As_{min} = \frac{0.25 + 0.052\ \lambda_{max}}{100} * b * t$$

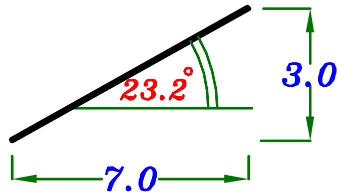
$$= \frac{0.25 + 0.052\ (13.0\ )}{100} * 350 * 700 = 2268 \ mm^2 > As_{total} \ \therefore \ O.K.$$

$$As = As` = \frac{As_{min}}{2} = 1134 \quad mm^2 \qquad 3 \phi 22$$


5Ø10\m'
5Ø12\m'
Sec. (1-1)
0.12
Sec. (2-2)
0.25
0.25
10Ø12
Sec. (3-3)
3Ø22
2Ø12
2Ø12
3Ø22
0.70
0.35

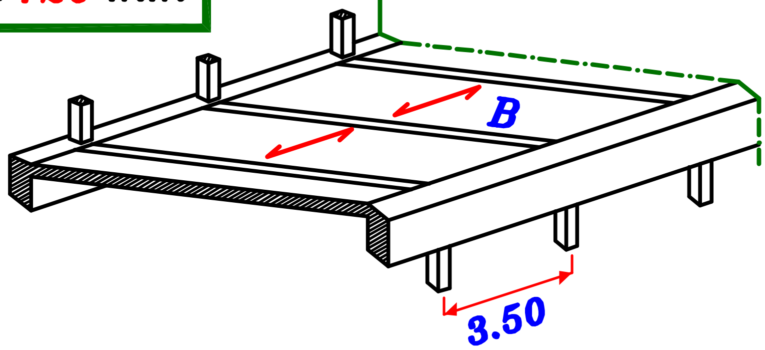
③ Design the Saw Tooth Slab.

Use Saw Tooth Beam Type Rested on the Arch Girder.



* **Loads on the Slab.** (One Way Slab)

$t_s = \frac{3500}{30} = 116$ mm   Take $\boxed{t_s = 120 \text{ mm}}$

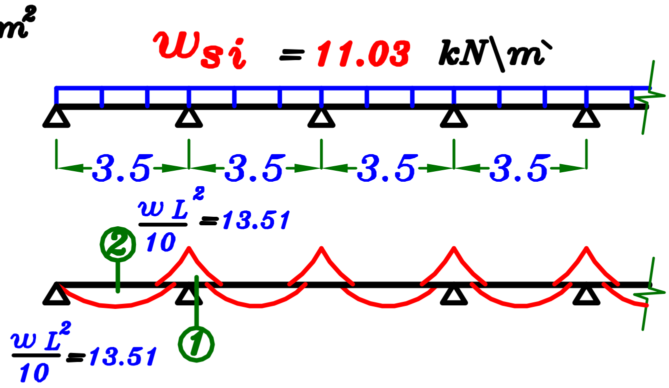


$(w_s)_{working} = 8.0$ kN\m² H.P.

$(w_s)_{U.L.} = 1.5 * 8.0$

$= 12.0$ kN\m² H.P.

$(w_{si}) = 12.0 * \cos 23.2^\circ = 11.03$ kN\m²



* **Design the Slab.**

شريحه افقيه فى بلاطه مائله

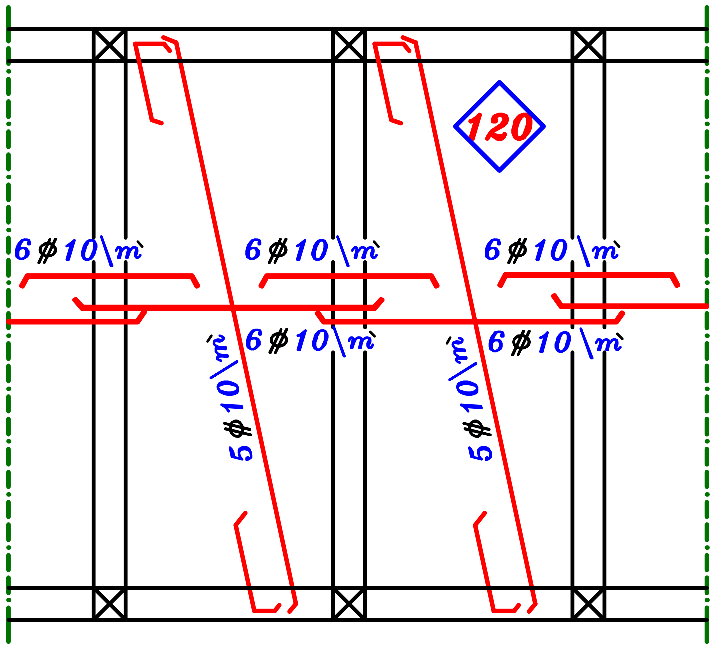
$M_{des.} = M \cos \alpha$

$= 13.51 * \cos 23.2^\circ = 12.4$ kN.m

Sec. ① $M_{U.L.} = 12.4$ kN.m\m` , $t_s = 120$ mm , $d = 120 - 20 = 100$ mm

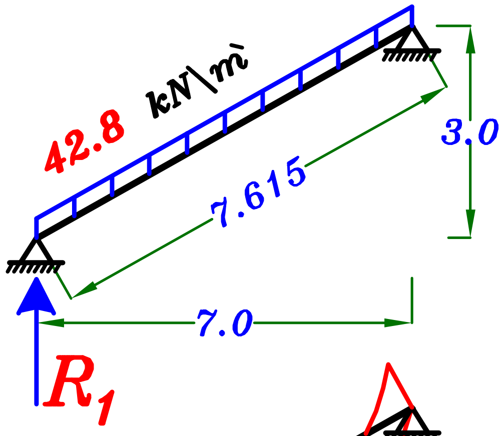
$100 = C_1 \sqrt{\frac{12.4 * 10^6}{25 * 1000}} \longrightarrow C_1 = 4.49 \longrightarrow J = 0.818$

$A_s = \frac{12.4 * 10^6}{0.818 * 360 * 100} = 421.1$ mm²\m   6 ϕ 10 \m`



* Design the Beam. B (250*700)

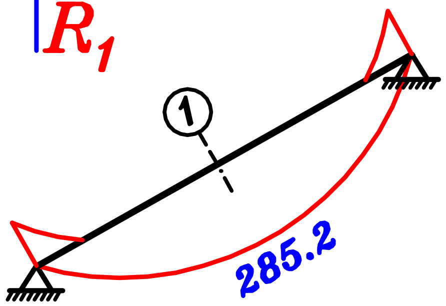
$w = O.W._{(beam)} + w_s * a \quad kN\backslash m'$



$w = 4.20 + 11.03 * 3.5 = 42.8 \quad kN\backslash m'$

$M = \dfrac{w\,L\,L'}{8} = 285.2 \; kN.m$

$R_1 = \dfrac{w\,L'}{2} = 162.96 \; kN$



Sec. ①

$M_{U.L.} = 285.2 \; kN.m \qquad T\text{-}Sec.$

$$B = \begin{cases} C.L. - C.L. = 3.5\; m = 3500 \; mm \\ 16\,t_s + b = 16 * 140 + 250 = 2490 \; mm \\ K\,\dfrac{L}{5} + b = 1.0 * \dfrac{7615}{5} + 250 = 1773 \; mm \end{cases} \qquad \boxed{B = 1773 \; mm}$$

$$\therefore\; 650 = C_1 \sqrt{\frac{285.2 * 10^6}{25 * 1773}} \longrightarrow C_1 = 8.10 \longrightarrow J = 0.826$$

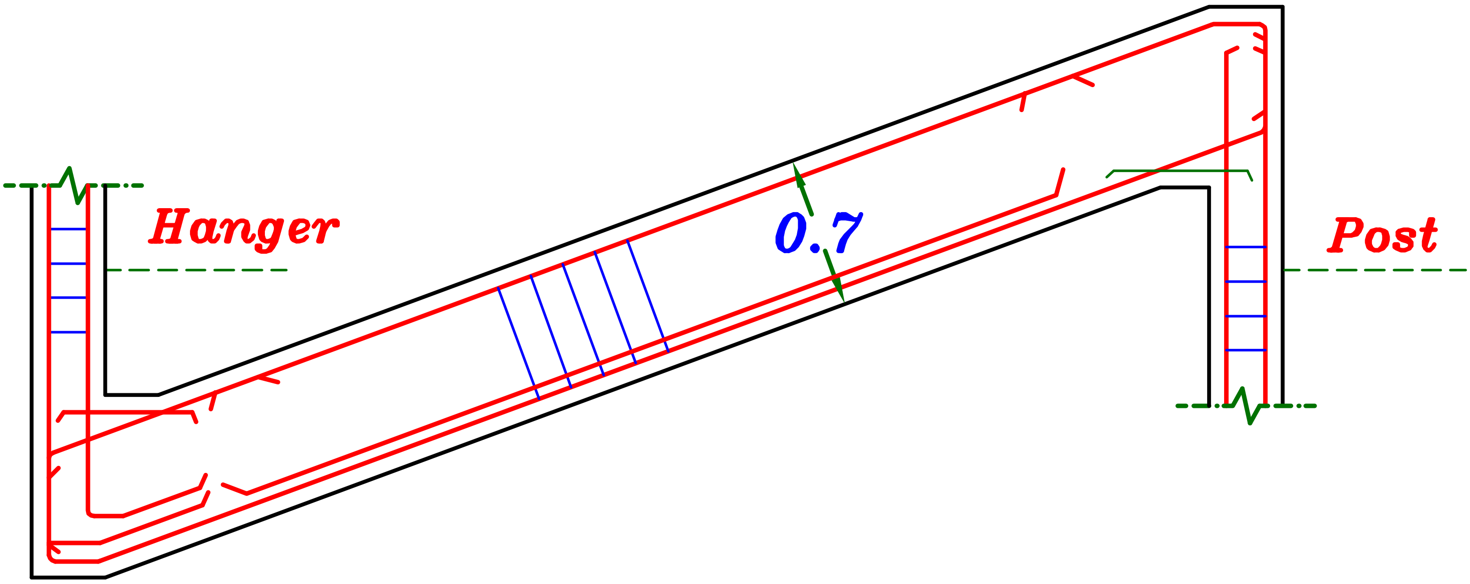
$$\therefore\; A_s = \frac{285.2 * 10^6}{0.826 * 360 * 650} = 1475.5 \; mm^2$$

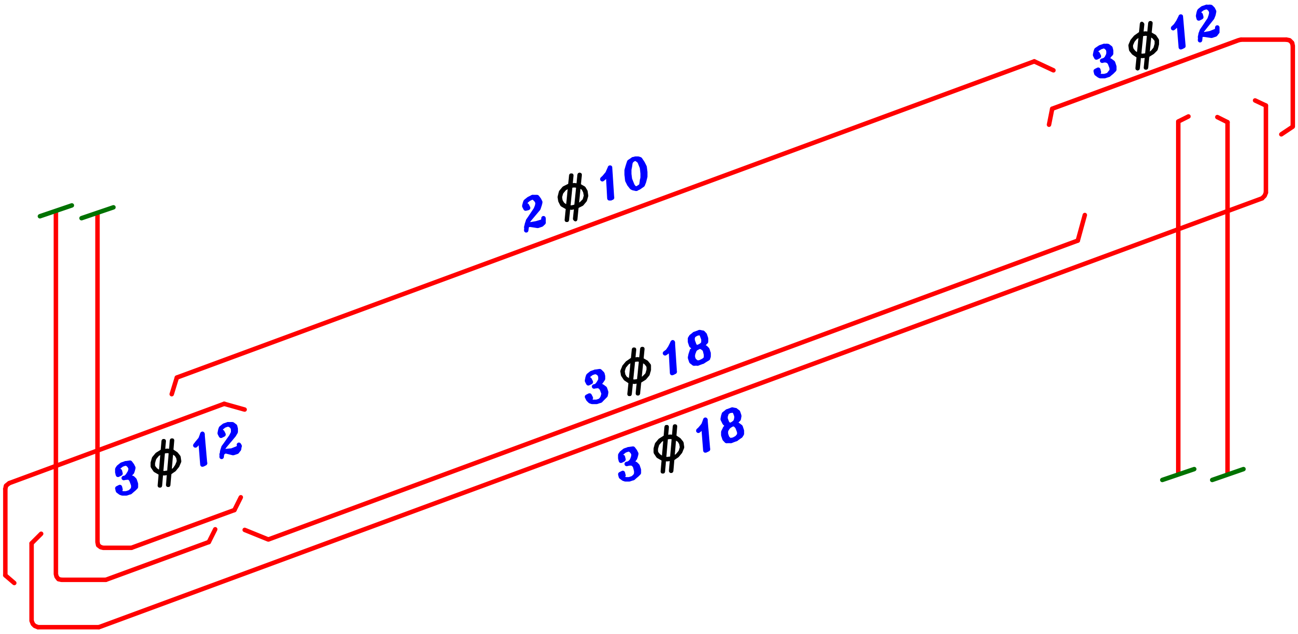
Check $A_{s\,min.}$ $\qquad A_{s_{req.}} = 1475.5 \; mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 650 = 507.8 \; mm^2$$

$\therefore\; A_{s_{req.}} > \mu_{min.}\, b\; d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1475.5 \; mm^2 \qquad 6\,\phi\,18$

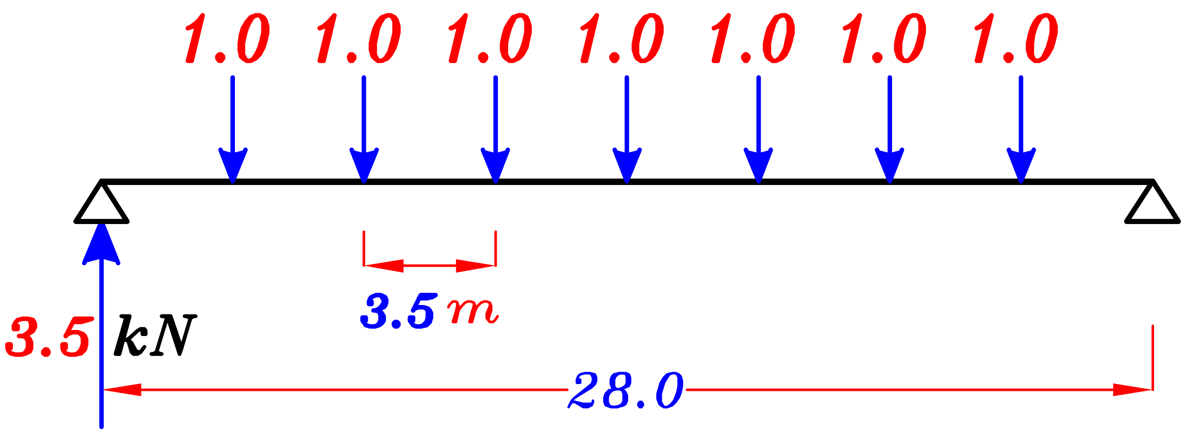
$$\therefore\; n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{18 + 25} = 5.23 = 5.0 \text{ bars}$$

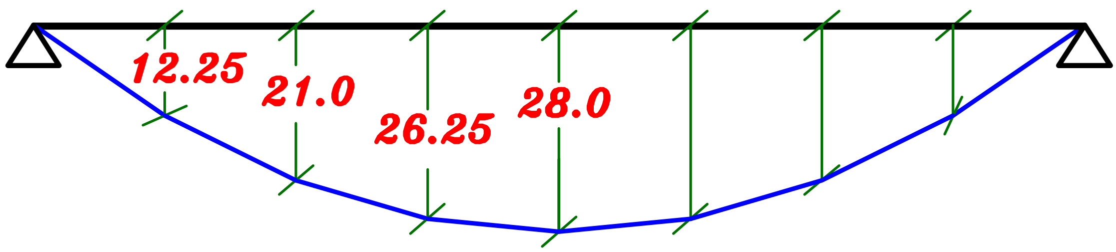

Hanger
0.7
Post


3 ϕ 12
2 ϕ 10
3 ϕ 18
3 ϕ 12
3 ϕ 18

4 Drawing Arch Girder.

Height of the Arch Girder



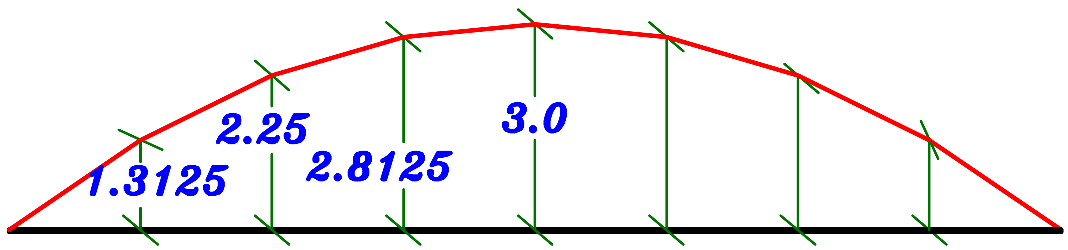


$M_\circ = 28.0$ kN.m $\longrightarrow$ $h_\circ = 3.0$ m

$M_1 = 26.25$ kN.m $\longrightarrow$ $h_1 = 2.8125$ m

$M_2 = 21.0$ kN.m $\longrightarrow$ $h_2 = 2.25$ m

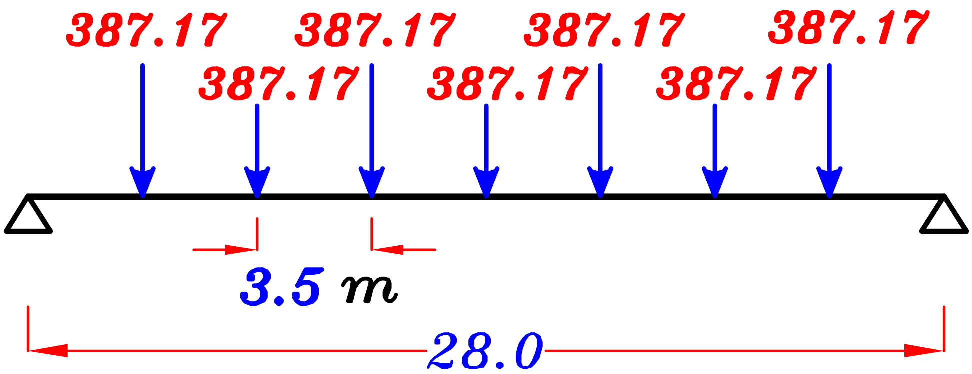
$M_3 = 12.25$ kN.m $\longrightarrow$ $h_3 = 1.3125$ m

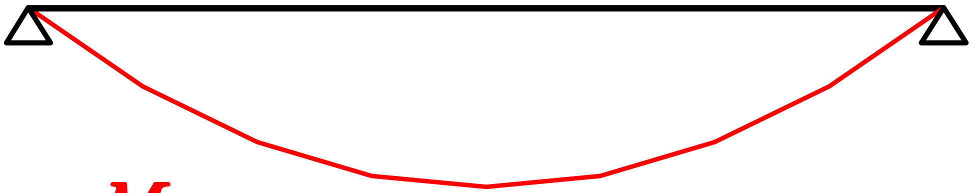


Loads on the Arch Girder.

Take o.w.(Arch) = 17.5 kN\m (U.L.)

$$F = 2\,R_1 + o.w.(Arch) * a = 2\,(162.96) + 17.5\,(3.5) = 387.17\ kN$$



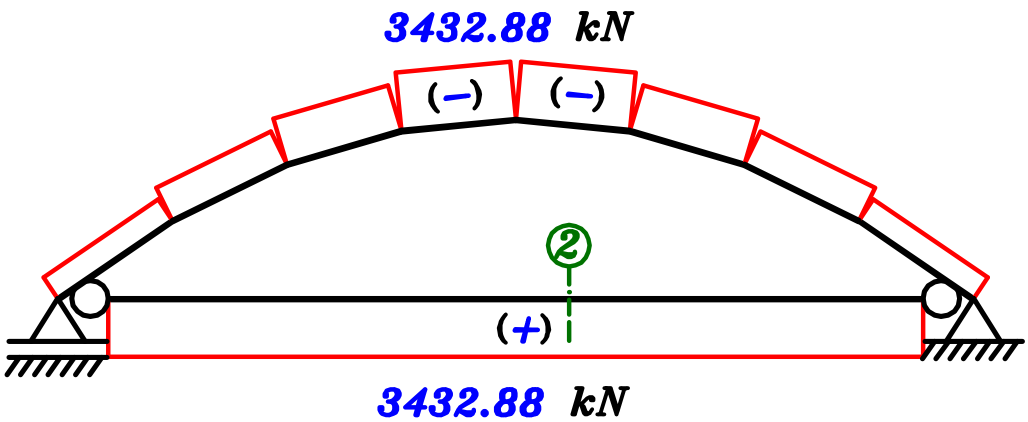


$$M_o = 387.17 * 28 = 10840.7\ kN.m$$

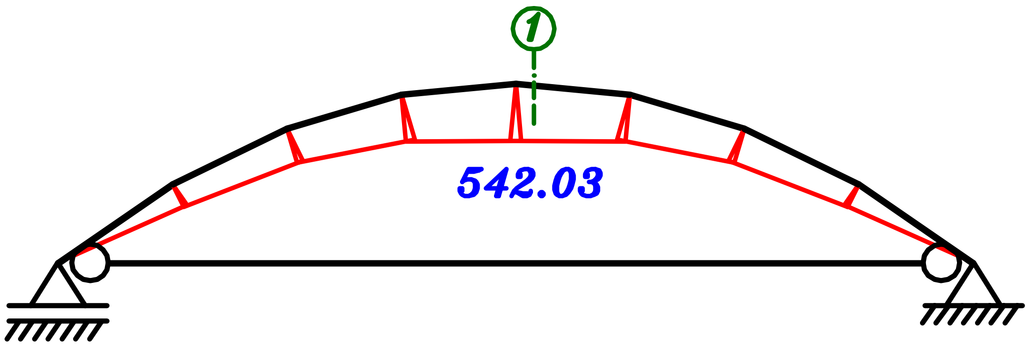
$$P = T = 0.95\,\frac{M_o}{h} = 0.95 * \frac{10840.7}{3.0} = 3432.88\ kN$$

$$M = 0.05\,M_o = 0.05\,(10840.7) = 542.03\ kN.m$$

N.F.D.



B.M.D.



* Design of Arch Girder.

Sec. ① $b = 0.35\ m$, $t = 1.20\ m$

$$P = 0.95\ \frac{M_o}{h} = 0.95 * \frac{10840.7}{3.0} = 3432.88\ kN$$

$$M = 0.05\ M_o = 0.05\ (10840.7) = 542.03\ kN.m$$

$$e = \frac{M}{P} = \frac{542.03}{3432.88} = 0.158\ m \quad \therefore \frac{e}{t} = \frac{0.158}{1.2} = 0.13 < 0.5 \xrightarrow{use} I.D.$$

$$\zeta = \frac{1.2 - 0.2}{1.2} = 0.83 \xrightarrow{use} \text{ECCS Design Aids Page } 4-24$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\ b\ t} &= \frac{3432.88 * 10^3}{25 * 350 * 1200} = 0.327 \\ \frac{M_U}{F_{cu}\ b\ t^2} &= \frac{542.03 * 10^6}{25 * 350 * 1200^2} = 0.043 \end{aligned}\right\} \rho = 1.0$$

$$As = As' = \mu * b * t = \rho * F_{cu} * 10^{-4} * b * t = 1.0 * 25 * 10^{-4} * 350 * 1200 = 1050\ mm^2$$

$$As_{total} = As + As' = 2100\ mm^2$$

$$-\ \text{Check}\quad As_{min.} = \frac{0.8}{100} * b * t = \frac{0.8}{100} * 350 * 1200 = 3360\ mm^2 > As_{total}$$

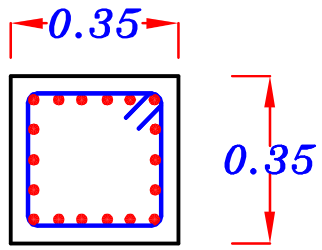
$$\therefore \text{take}\ As = As' = \frac{As_{min.}}{2} = \frac{3360}{2} = 1680\ mm^2 \qquad 5\phi22$$

* Design of Tie.

Sec. ② $(350 * 350)$

Neglect o.w. of Tie.

18∅28



Sec. (2-2)

$$T = 0.95\ \frac{M_o}{h} = 0.95 * \frac{10840.7}{3.0} = 3432.88\ kN$$

$$As = \frac{T}{F_y \backslash \gamma_s} = \frac{3432.88 * 10^3}{360 \backslash 1.15} = 10966\ mm^2 \qquad 18\phi28$$

* **Design of the hangers.** (250 * 250)

Take o.w.(hanger) = 3.50 kN (U.L.)

$T = o.w.(hanger) + R_1$

$= 3.50 + 162.96 = 166.46$ kN

$$A_s = \frac{T}{F_y \backslash \gamma_s} = \frac{166.46 * 10^3}{360 \backslash 1.15} = 531.7 \ mm^2$$ 4 ϕ 16

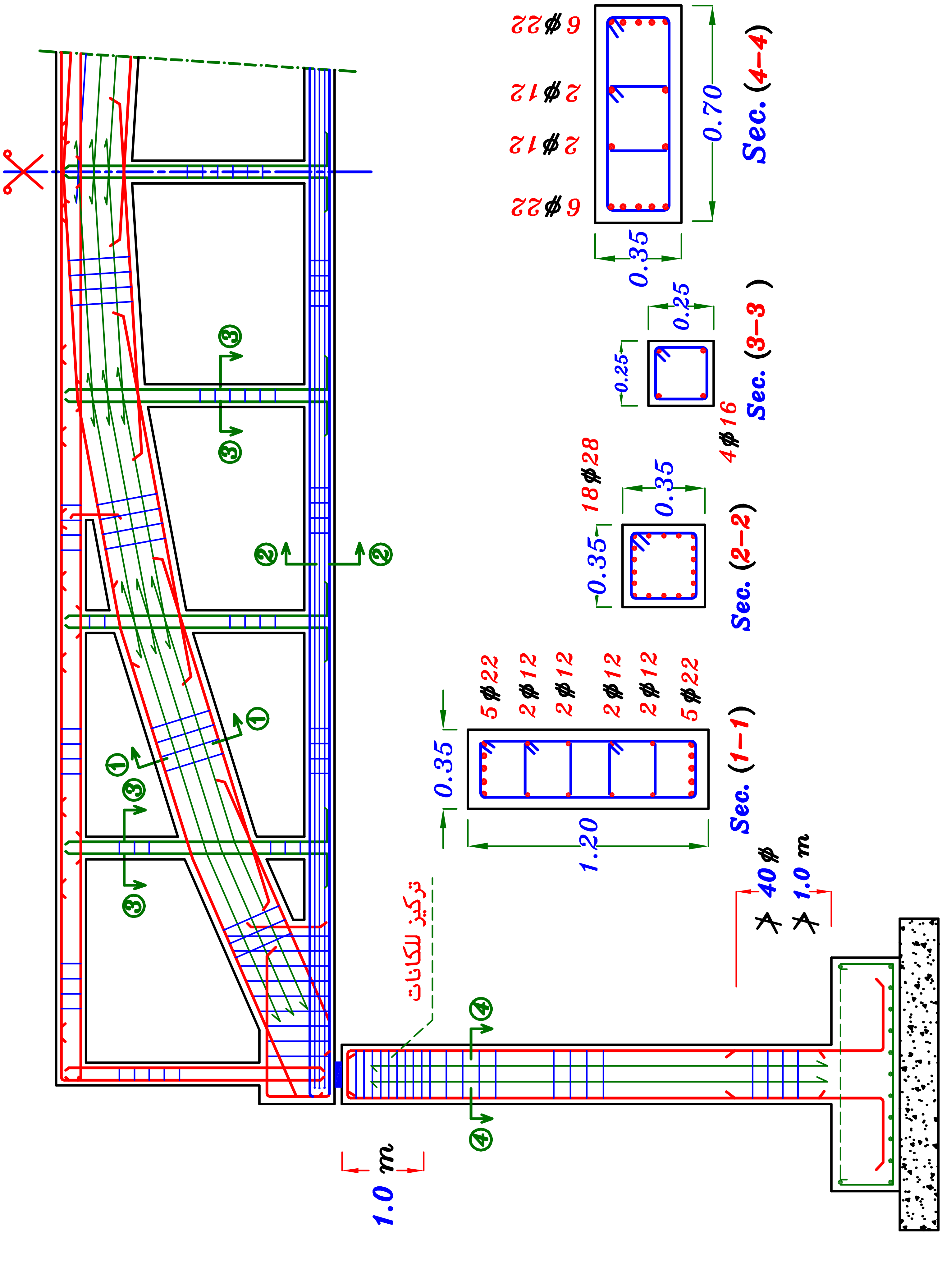
* **Design the Post.** (250 * 250)

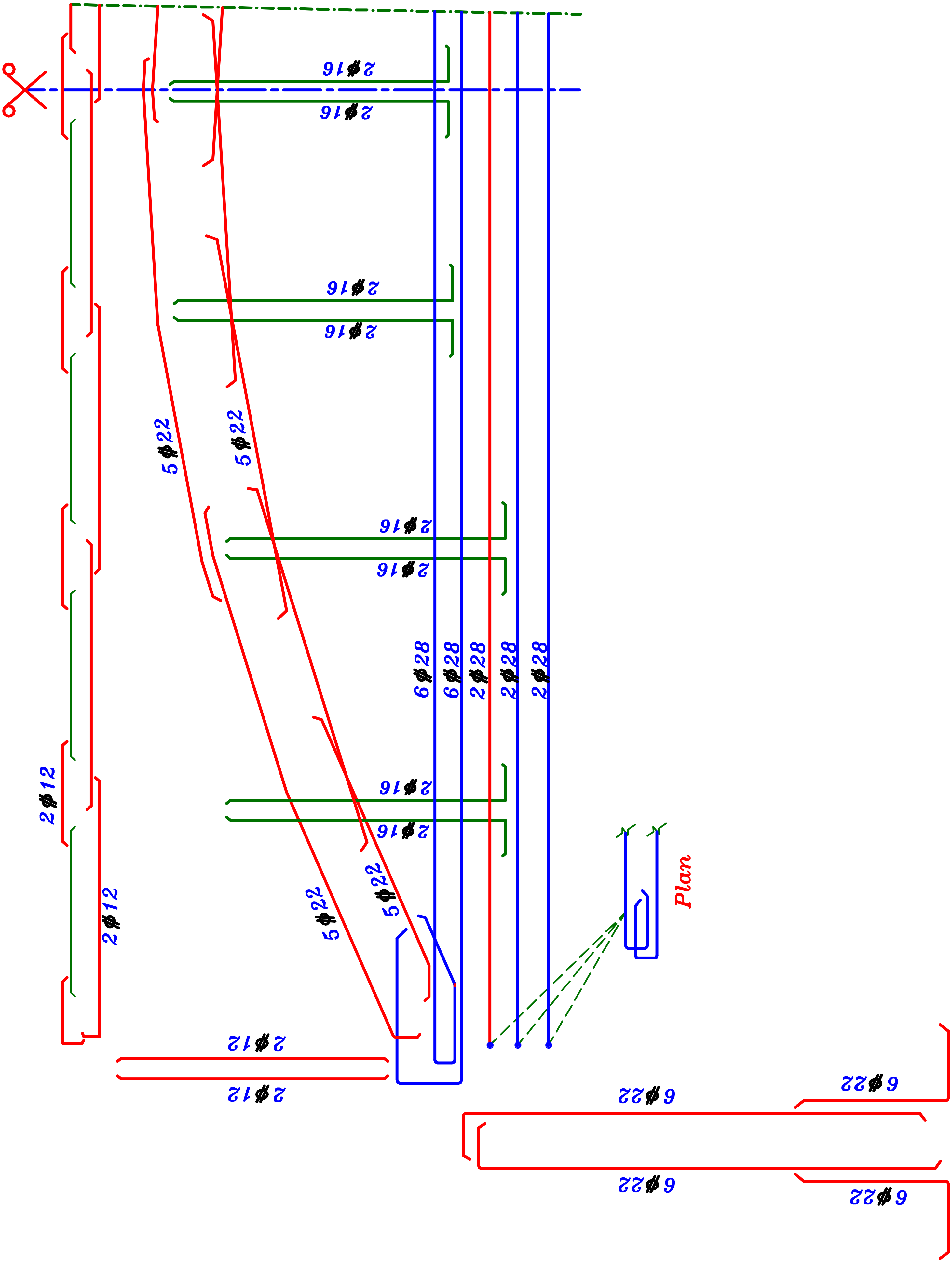
Take o.w.(Post) = 3.50 kN (U.L.)

$P = o.w.(Post) + R_1$

$= 3.50 + 162.96 = 166.46$ kN

$A_s =$ 4 ϕ 12


Sec. (1-1)
Sec. (2-2)
Sec. (3-3)
Sec. (4-4)
5 ϕ 22
2 ϕ 12
2 ϕ 12
2 ϕ 12
2 ϕ 12
5 ϕ 22
18 ϕ 28
4 ϕ 16
6 ϕ 22
2 ϕ 12
2 ϕ 12
6 ϕ 22
0.35
1.20
0.35
0.35
0.25
0.25
0.35
0.70
40 ϕ
1.0 m
1.0 m
تركيز للكانات


2 ϕ 16
5 ϕ 22
6 ϕ 28
2 ϕ 28
2 ϕ 12
6 ϕ 22
Plan

Example.

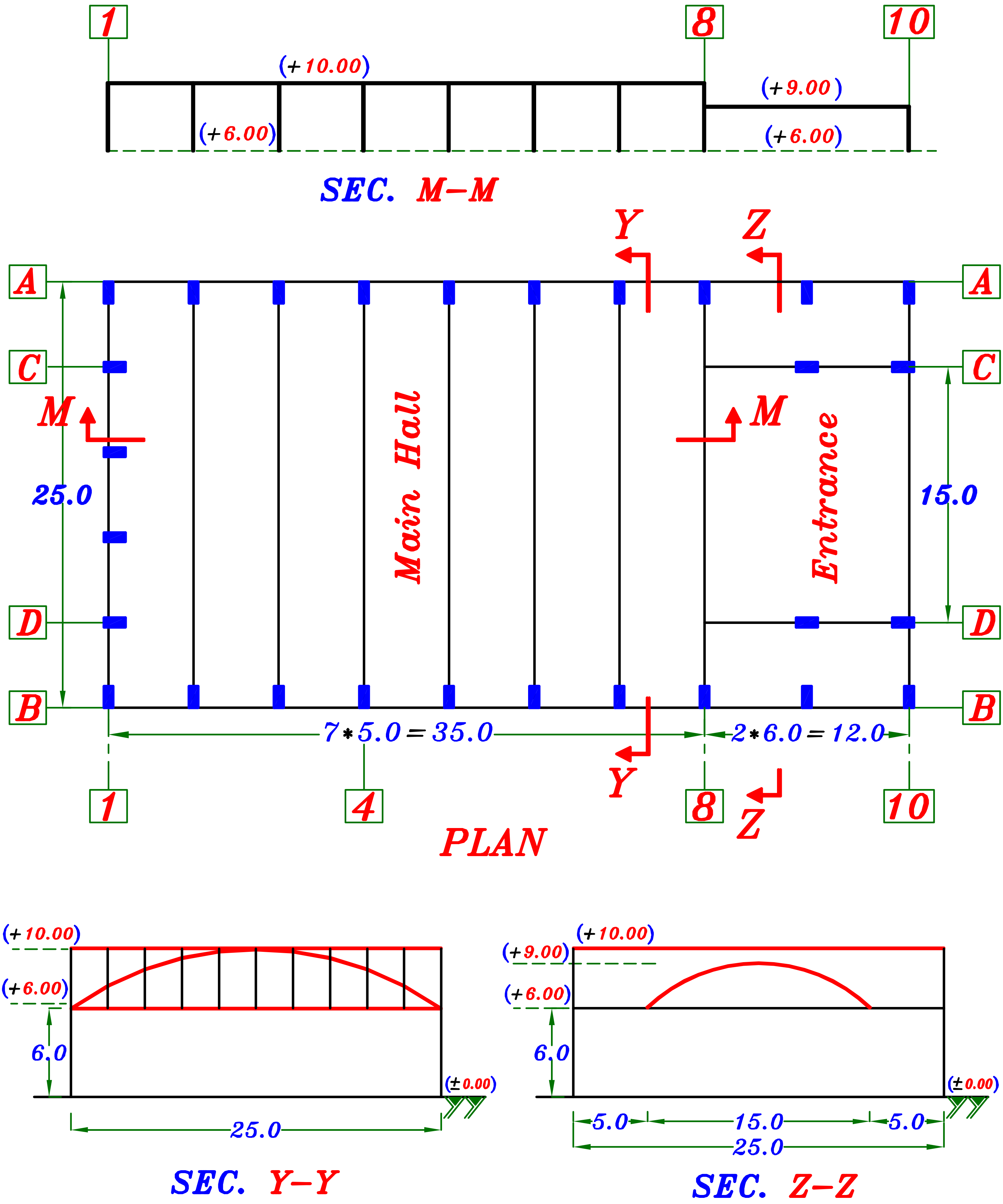


Figure (1)

Figure (1) shows plan and sectional elevations of an Exhibition Hall.
*The main Hall area is 25.0 * 35.0 m with no interior columns allowed.*
Levels and shape of the covering R.C. roof are as shown in cross sections Y−Y , Z−Z & M−M. Columns are only allowed where shown in plan.

− $F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

It is required to :

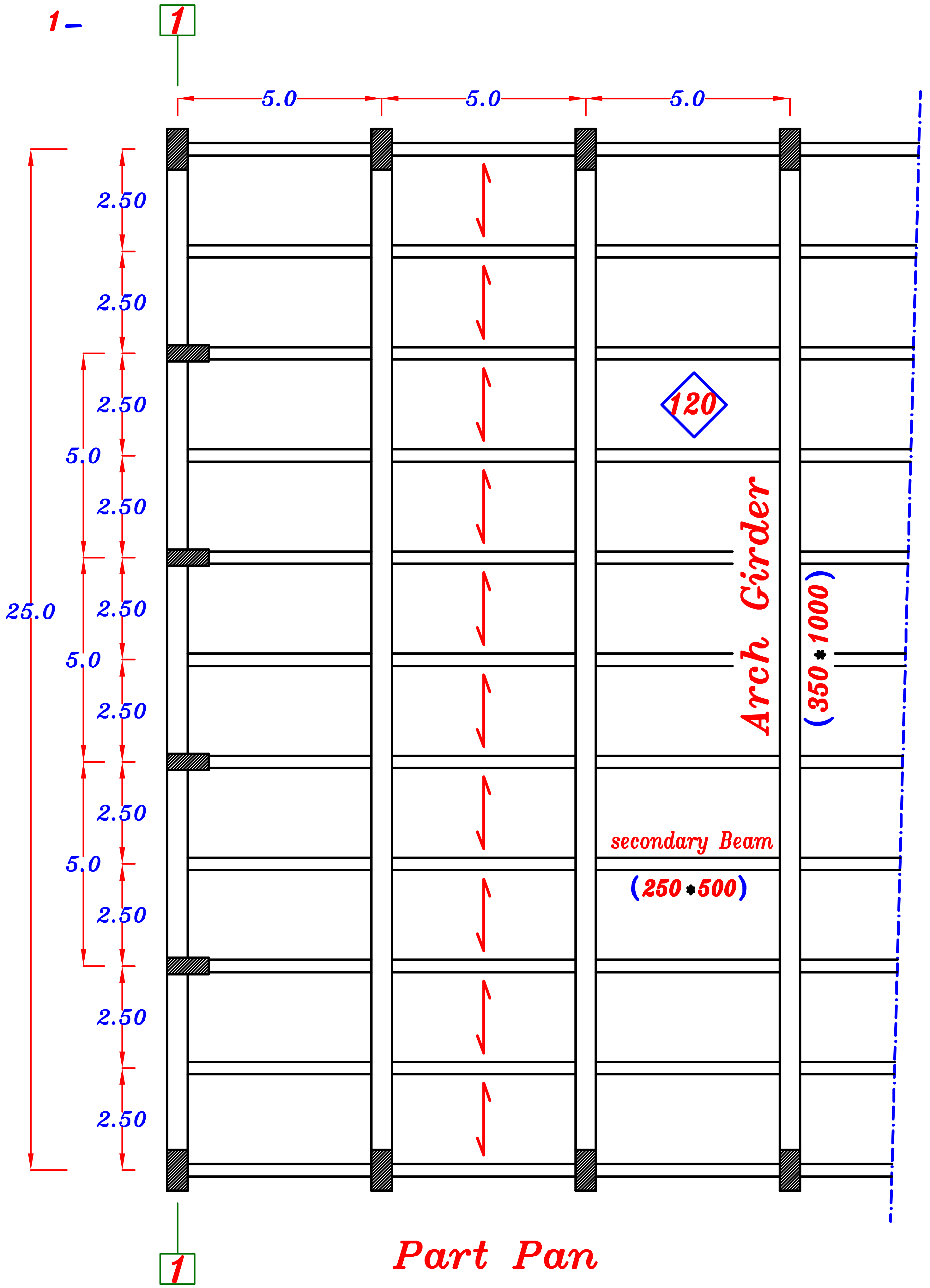
1 – *Choose a convenient main system (main supporting element) For the main Hall and without any calculation but with reasonably assumed concrete dimensions, Draw to a scale 1:50 in elevation and part plan the main supporting element of the Main Hall and its roof slab systems.*

2 – *Design all slabs and draw their details of reinforcement in plan and cross−sections (IF needed) to a scale 1:50 This should include :*

a) *Arched slab of the Entrance between axes 8 & 10 , C & D*
b) *Horizontal slabs of the roof of the Main Hall.*

3 – *Design the Main supporting element of Axis 8−8 of the Main Hall area.*

4 – *Draw to a scale 1:50 in elevation and cross section details of reinforcement of the designed main supporting element.*

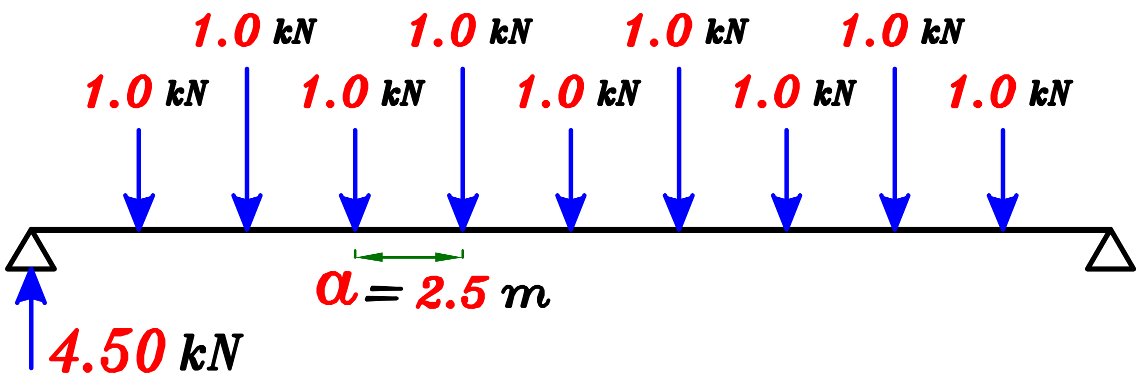

1 -
1
5.0
5.0
5.0
2.50
2.50
2.50
2.50
2.50
2.50
2.50
2.50
2.50
2.50
5.0
5.0
5.0
25.0
120
Arch Girder
(350 * 1000)
secondary Beam
(250 * 500)
1

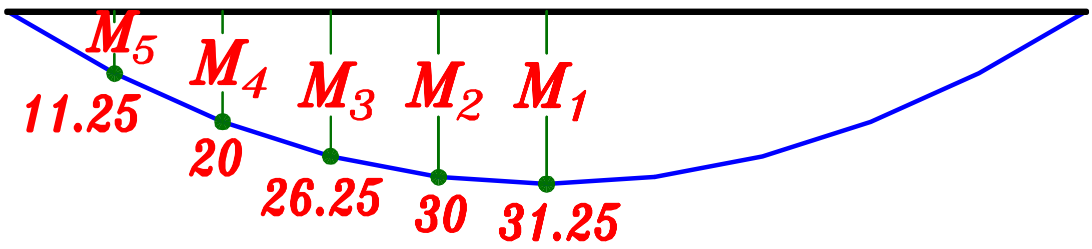
Part Pan

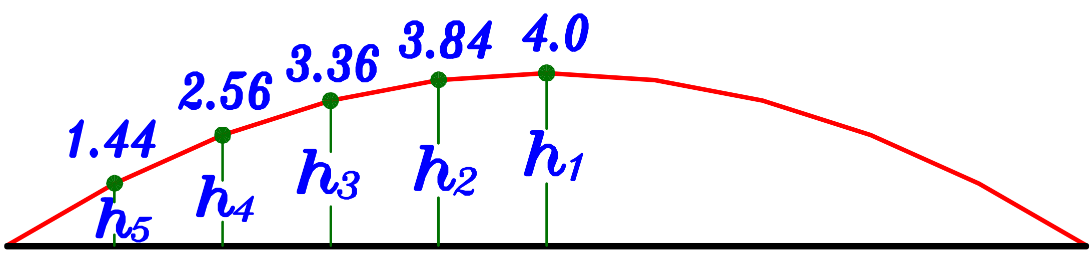
Drawing Arch Girder.

Arch Girder في وسط القاعه و الاحمال عليه متساويه
لذا يمكن تحديد ارتفاعاته أولا بوضع أحمال 1 kN عند كل Joint
و أخذ نفس النسب بين الارتفاعات هى نفس النسب بين العزوم

Take the Arch Girder 10 segments $a = 2.5\ m$







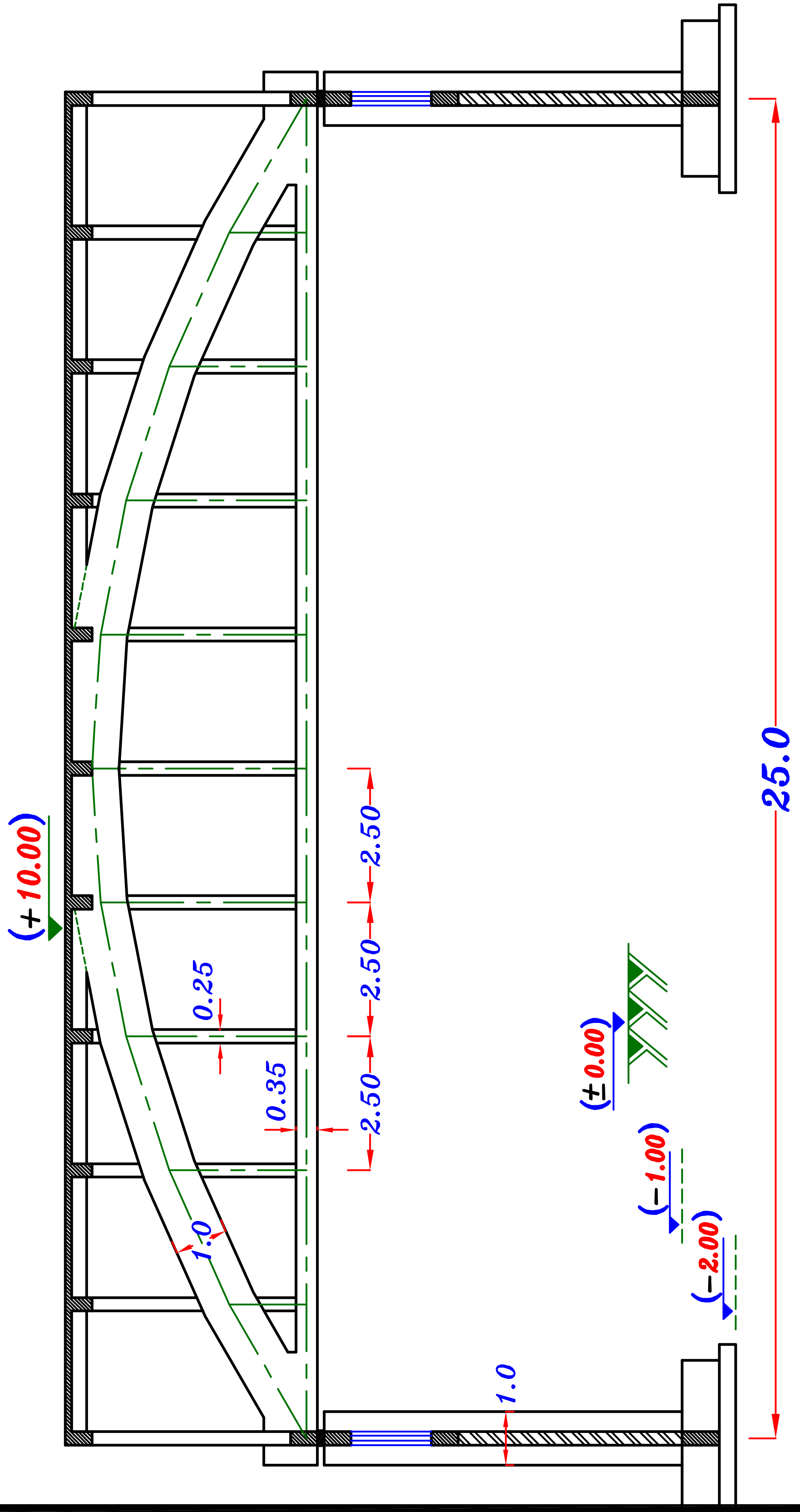
$M_1 = 31.25\ kN.m \longrightarrow h_1 = 4.0\ m$

$M_2 = 30.0\ kN.m \longrightarrow h_2 = 3.84\ m$

$M_3 = 26.25\ kN.m \longrightarrow h_3 = 3.36\ m$

$M_4 = 20.0\ kN.m \longrightarrow h_4 = 2.56\ m$

$M_5 = 11.25\ kN.m \longrightarrow h_5 = 1.44\ m$


(+10.00)
25.0
2.50
2.50
2.50
0.25
0.35
1.0
1.0
(±0.00)
(−1.00)
(−2.00)

② Design the Arch Slab.

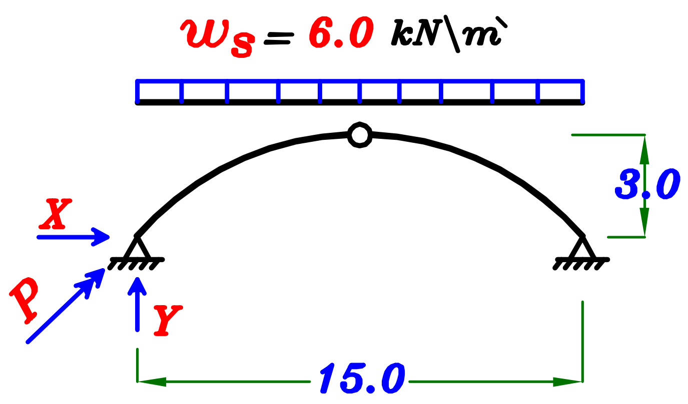
Take $t_s = 120$ mm

$\because$ F.C. + L.L. $= 1.0$ kN\m^2 H.P.

$$(w_s)_{U.L.} = 1.5\,(t_s\,\gamma_c + F.C. + L.L.)$$

$$(w_s)_{U.L.} = 1.5\,(0.12 * 25 + 1.0)$$

$$= 6.0 \text{ kN\\m}^2 \quad (H.P.)$$



To Get N.F.

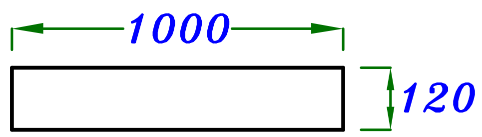
$$Y = \frac{w\,L}{2} = \frac{6.0 * 15}{2} = 45.0 \text{ kN\\m}$$

$$X = \frac{w\,L^2}{8\,h} = \frac{6.0 * 10^2}{8 * 3.0} = 56.25 \text{ kN\\m}$$

$$P = \sqrt{X^2 + Y^2} = \sqrt{56.25^2 + 45.0^2} = 72.0 \text{ kN}$$

* Design the Arch Slab.

Neglect B.M. & Design on N.F. only.



$\therefore$ Designed as a Column.

$$\therefore P_{U.L.} = 0.35\,A_c\,F_{cu} + 0.67\,A_s\,F_y$$

Take $A_c = 120 * 1000 = 120000$ mm^2

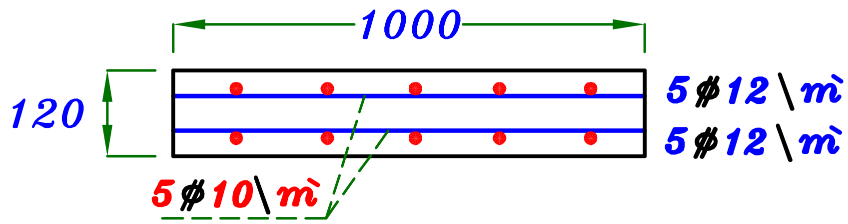
$$\therefore 72.0 * 10^3 = 0.35\,(120000)(25) + 0.67\,A_s\,(360)$$

$$\therefore A_s = -\,4054 \text{ mm}^2 = (-\,Ve) \text{ Value}$$

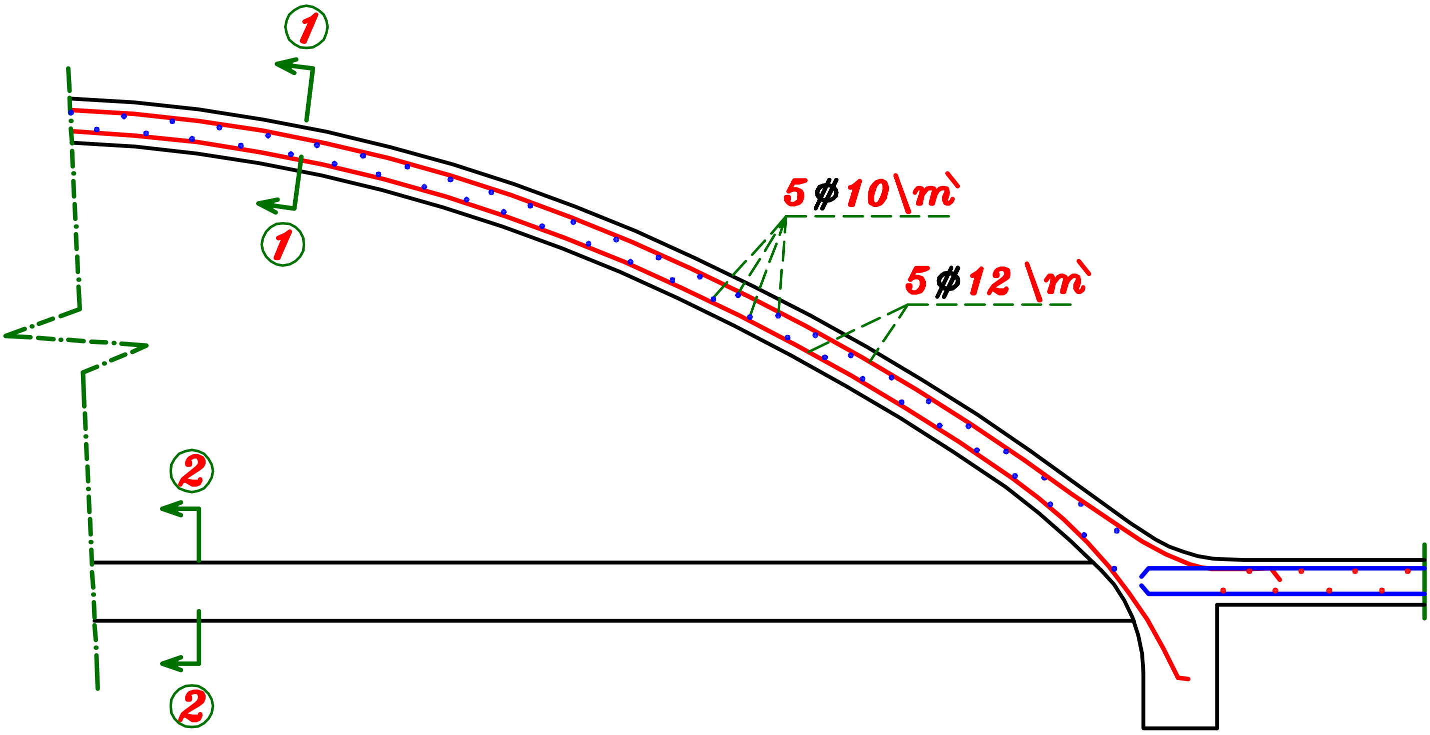
$$\therefore \text{Take } A_s = A_{s\,min.} = \frac{0.8}{100} * b * t$$

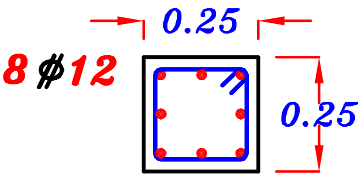
$$\therefore A_s = \frac{0.8}{100} * 120 * 1000 = 960 \text{ mm}^2 = A_{s\,total}$$

$$\therefore \text{Upper Steel \& Lower Steel} = \frac{A_{s\,total}}{2} = \frac{960}{2} = 480 \text{ mm}^2$$

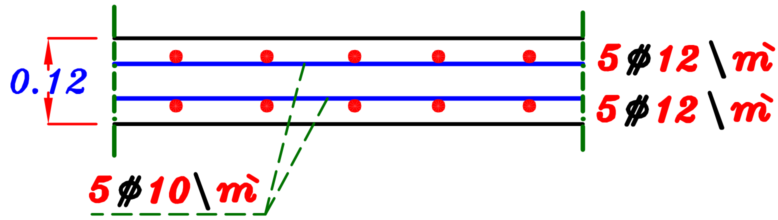


5 φ12 \m`





Sec. (2-2)



Sec. (1-1)

Design the Solid Slab rested on the Arch Girder.

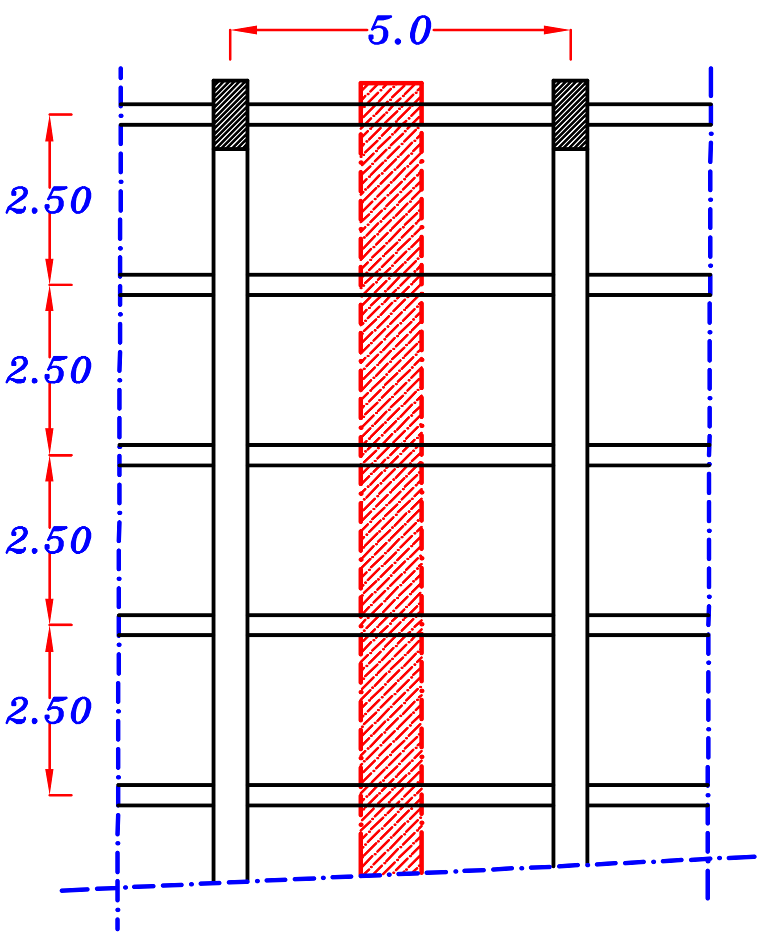
Use one way solid slab.

$$t_s = \frac{2500}{30} = 83.3 \ \text{mm}$$

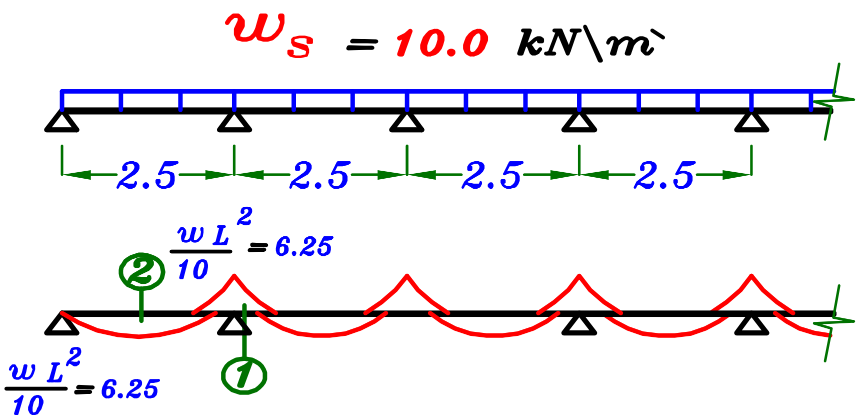
$$\boxed{t_s = 120 \ \text{mm}}$$

Take $F.C. = 3.0 \ kN\backslash m^2$

$L.L. = 1.0 \ kN\backslash m^2$



$$(w_s)_{U.L.} = 1.4\,(0.12 * 25 + 3.0) + 1.6\,(1.0) = 10.0 \ kN\backslash m^2$$



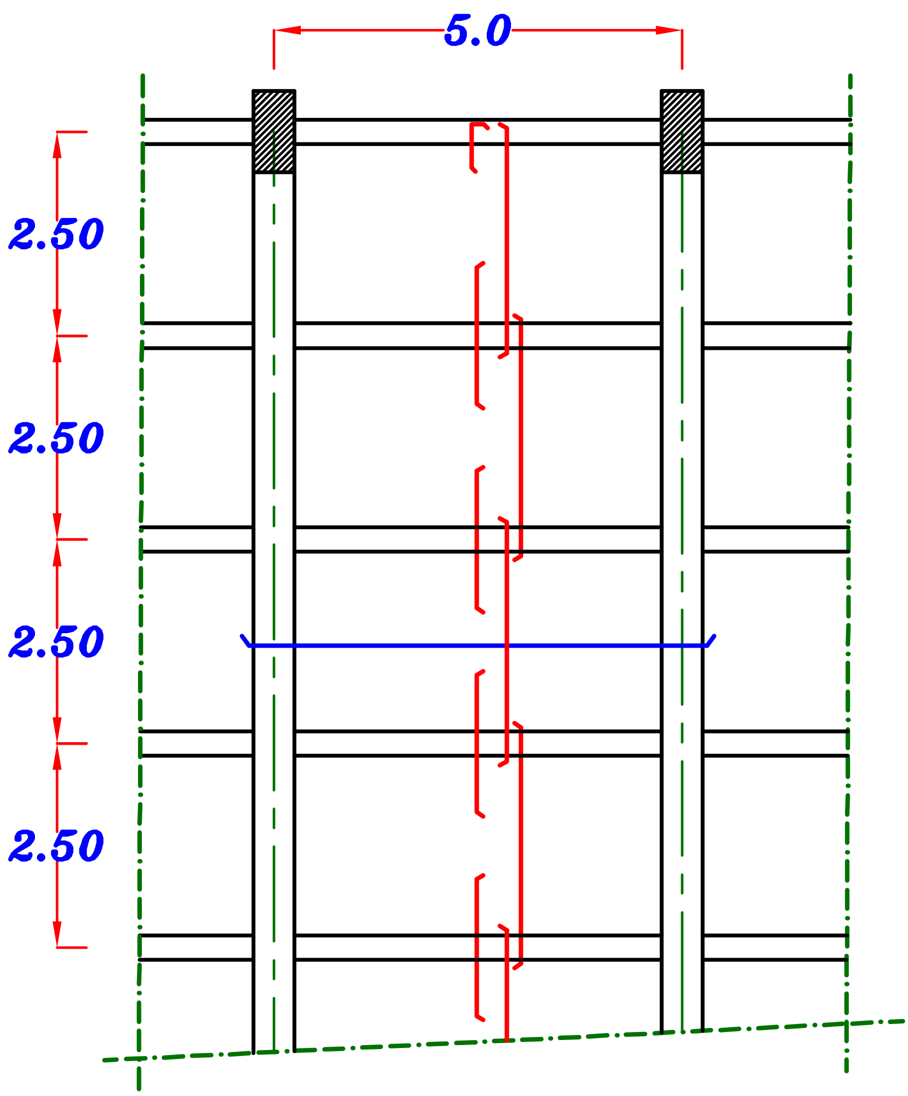
Sec. ①

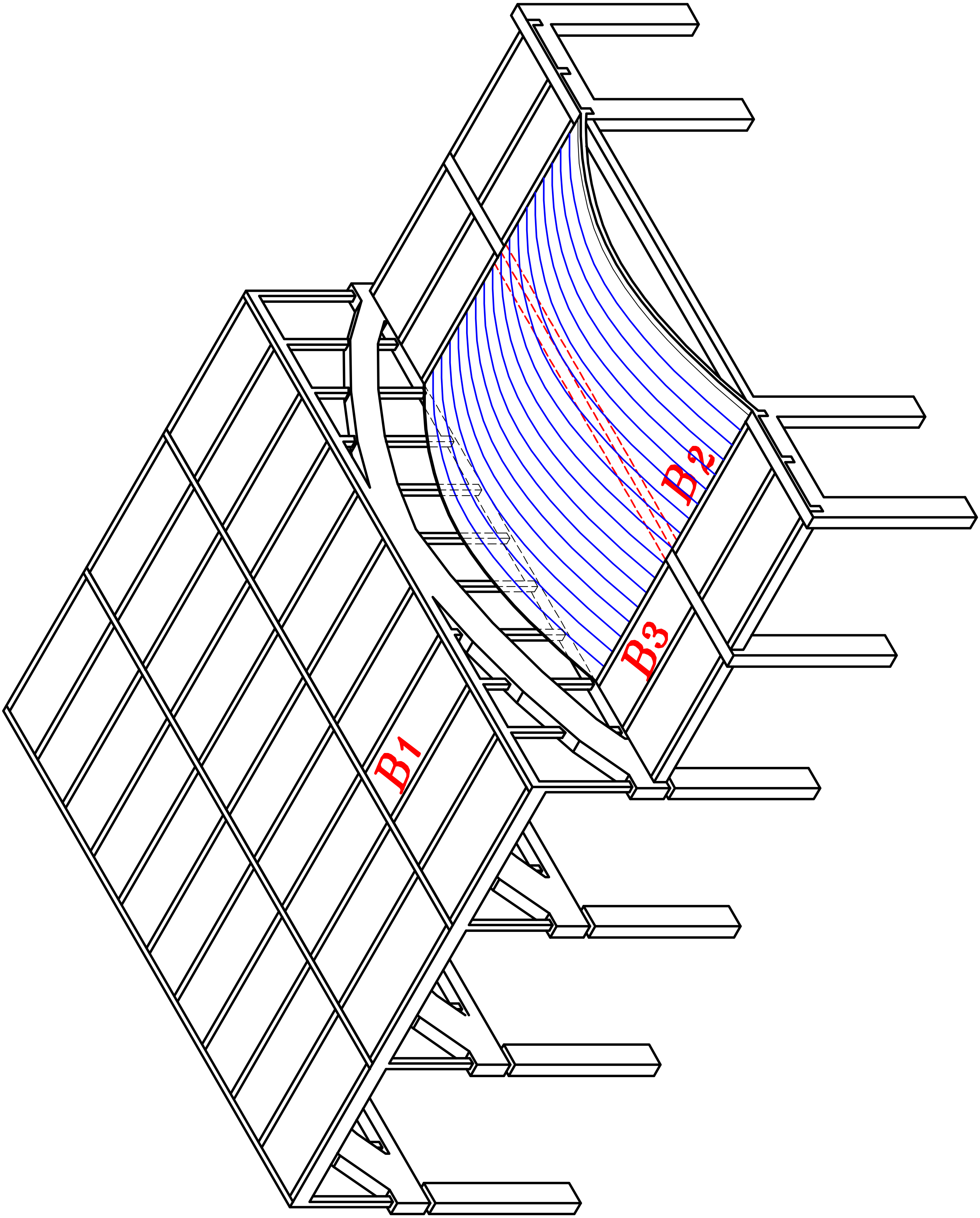
$M_{U.L.} = 6.25 \ kN.m\backslash m'$, $t_s = 120 \ mm$, $d = 120 - 20 = 100 \ mm$

$$100 = C_1 \sqrt{\frac{6.25 * 10^6}{25 * 1000}} \longrightarrow C_1 = 6.32 \longrightarrow J = 0.826$$

$$A_s = \frac{6.25 * 10^6}{0.826 * 360 * 100} = 210.1 \ mm^2\backslash m$$

$5 \phi 10 \backslash m'$




B1
B2
B3

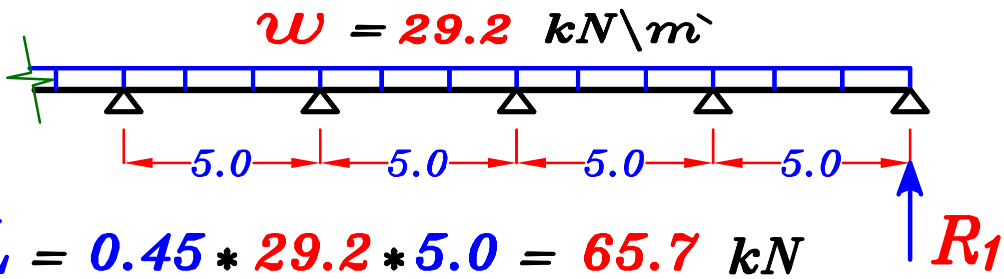
* Loads on the Arch Girder at axis 8-8.

* Loads on Beam B_1

$w_S = 10.0 \ kN\backslash m^2$

$w = o.w. + w_S * a = 4.2 + 10 * 2.5 = 29.2 \ kN\backslash m$

لان المطلوب تصميم Arch Girder عند axis (8)
لذلك سنحتاج أول reaction للكمره B_1



$R_1 = 0.45 \ w \ L = 0.45 * 29.2 * 5.0 = 65.7 \ kN$

$\boxed{R_1 = 65.7 \ kN}$

Loads on Beam B_2

$\boxed{X = 56.25 \ kN\backslash m}$, $\boxed{Y = 45.0 \ kN\backslash m}$ From the Arch Slab

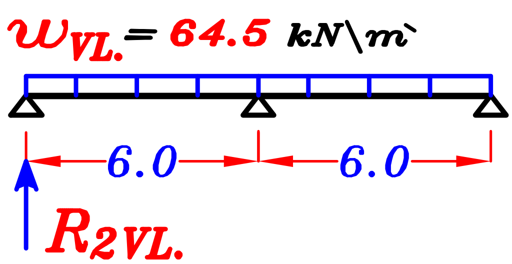
VL. Beam

$w_{VL.} = o.w. + Y + w_S * \frac{a}{2} = 7.0 + 45.0 + 10 * 1.25 = 64.5 \ kN\backslash m$

لان المطلوب تصميم Arch Girder عند axis (8)
لذلك سنحتاج أول reaction للكمره B_2

$R_{2VL.} = 0.40 \ w \ L$

$= 0.40 * 64.5 * 6.0 = 154.8 \ kN$



$\boxed{R_{2VL.} = 154.8 \ kN}$

HL. Load

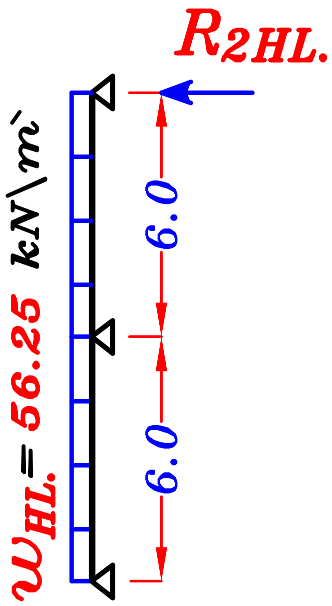
$w_{HL.} = X = 56.25$ kN\m

لحساب ال Tension على ال Tie سنحتاج أول reaction

$R_{2HL.} = 0.40\, w\, L$

$= 0.40 * 56.25 * 6.0 = 135$ kN

$\boxed{R_{2HL.} = 135.0 \text{ kN}}$



* **Loads on Beam B_3**

$w = o.w. + w_s * a = 4.2 + 10 * 2.5 = 29.2$ kN\m

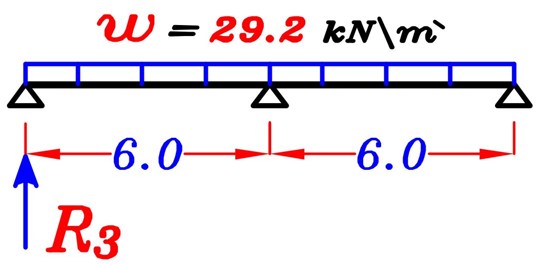
لان المطلوب تصميم Arch Girder عند 8 axis

لذلك سنحتاج أول reaction للكمره B_3

$R_3 = 0.40\, w\, L$

$= 0.40 * 29.2 * 6.0 = 70.1$ kN

$\boxed{R_3 = 70.1 \text{ kN}}$



Loads on Arch Girder at axis 8-8

Take $o.w.(Arch) = 17.5$ kN\m` (U.L.)

$$F_1 = R_1 + o.w.(Arch) * a$$

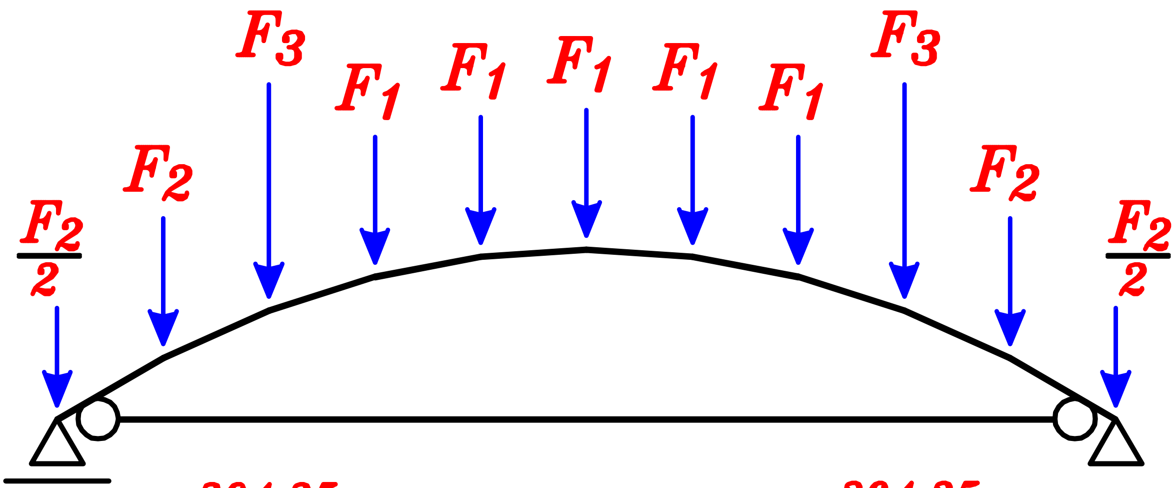
$$= 65.7 + 17.5\,(2.5) = 109.45 \text{ kN}$$

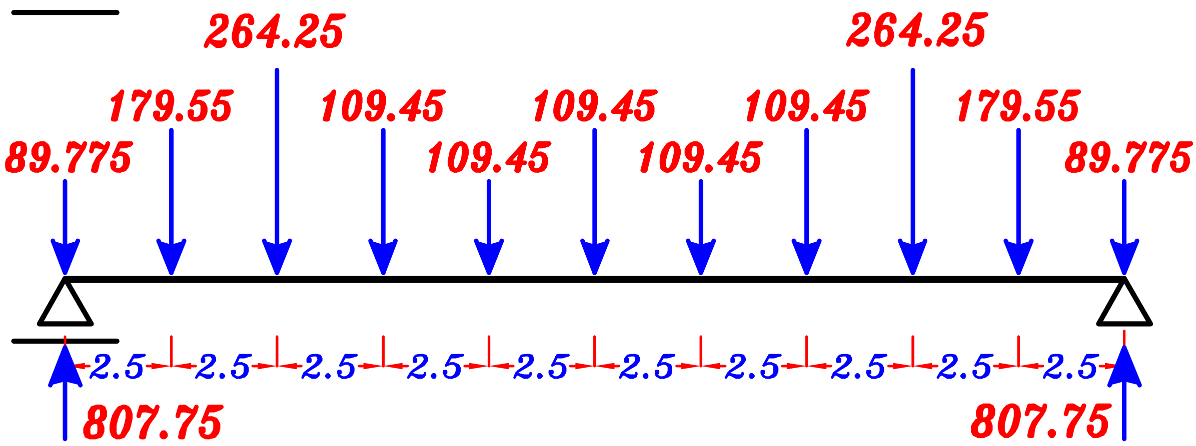
$$F_2 = R_1 + R_3 + o.w.(Arch) * a$$

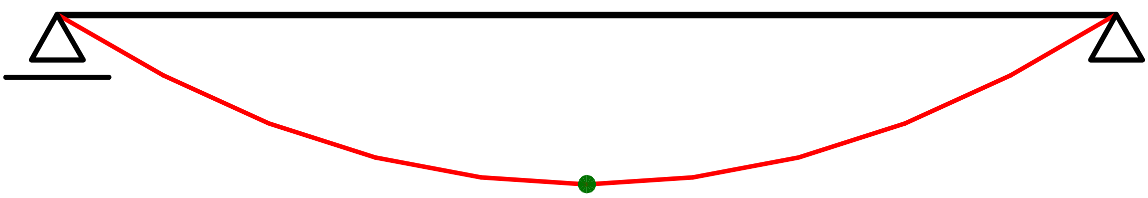
$$= 65.7 + 70.1 + 17.5\,(2.5) = 179.55 \text{ kN}$$

$$F_3 = R_1 + R_{2\,VL.} + o.w.(Arch) * a$$

$$= 65.7 + 154.8 + 17.5\,(2.5) = 264.25 \text{ kN}$$





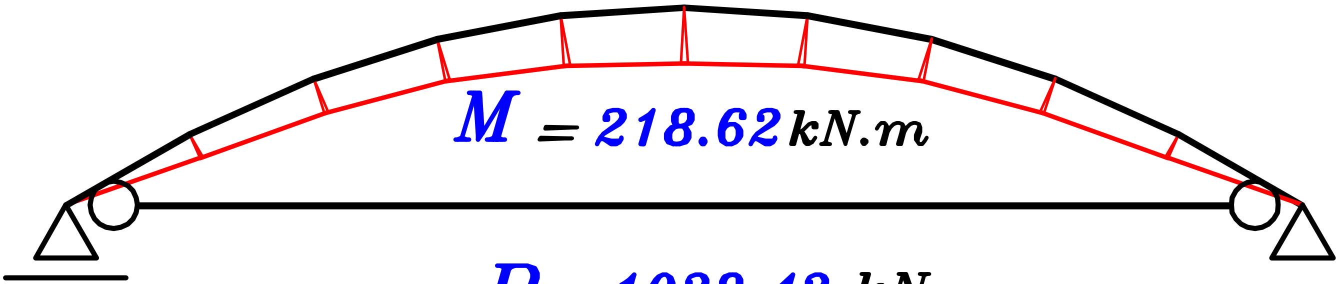


$$M_o = 4372.3 \text{ kN.m}$$

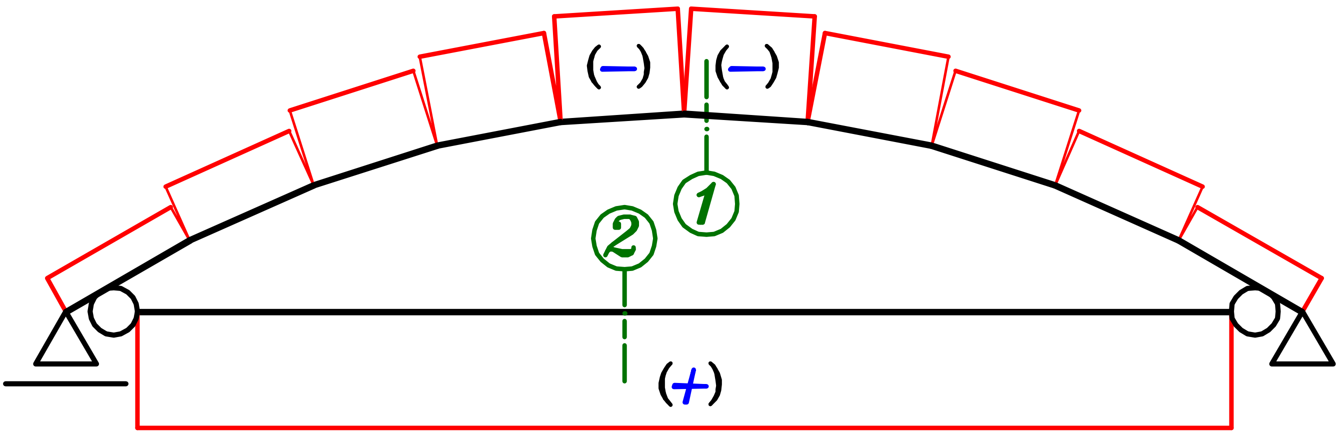
$$P = 0.95\,\frac{M_o}{h} = 0.95 * \frac{4372.3}{4.0} = 1038.42 \text{ kN}$$

$$T = 0.95\,\frac{M_o}{h} + R_{2HL.} = 1038.42 + 135.0 = 1173.42 \text{ kN}$$

$$M = 0.05\ M_o = 0.05\,(4372.3) = 218.62 \text{ kN.m}$$



P = 1038.42 kN



T = 1173.42 kN

Sec. 1 **Neglect Effect of Buckling**

$b = 0.35$ m , $t = 1.0$ m , $M = 218.62$ kN.m , $P = 1038.42$ kN

$$e = \frac{M}{P} = \frac{218.62}{1038.42} = 0.21 \text{ m} \qquad \therefore \frac{e}{t} = \frac{0.21}{1.0} = 0.21 < 0.5 \xrightarrow{use} \text{I.D.}$$

$$\zeta = \frac{1.0 - 0.1}{1.0} = 0.9 \xrightarrow{use} \text{ECCS Design Aids Page } 4\text{–}23$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{1038.42 * 10^3}{25 * 350 * 1000} = 0.118 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{218.62 * 10^6}{25 * 350 * 1000^2} = 0.025 \end{aligned}\right\}\ \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 25 * 10^{-4} = 2.5 * 10^{-3}$$

$$A_s = A_s' = \mu * b * t = 2.5 * 10^{-3} * 350 * 1000 = 875 \ mm^2$$

$$A_{s\,Total} = A_s + A_s' = 2 * 875 = 1750 \ mm^2$$

– Check $A_{s\,min.} = \frac{0.8}{100} * b * t = \frac{0.8}{100} * 350 * 1000 = 2800 > A_{s\,Total}$

Take $A_s = A_s' = \frac{A_{s\,min}}{2} = \frac{2800}{2} = 1400 \ mm^2$ 6 ϕ 18

* **Design of Tie.**

Sec. ② (350 * 350) $T = 1173.42$ kN

$\boxed{\text{Negelct o.w.(Tie)}}$

$$A_s = \frac{T}{F_y \backslash \gamma_s} = \frac{1173.42 * 10^3}{360 \backslash 1.15} = 3731.8 \ mm^2$$ 8 ϕ 25

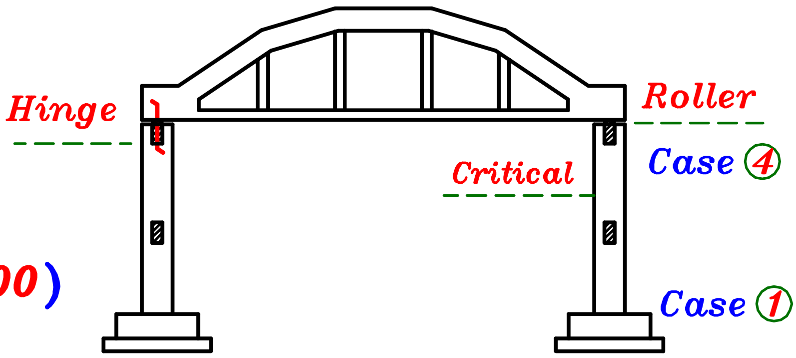
* **Design of Hanger.** (250 * 250) 4 ϕ 12

* **Design the Post.** (250 * 250)

Take o.w.(Post) = 3.50 kN (U.L.)

$P = o.w.(Post) + R_1 = 3.50 + 65.7 = 69.2$ kN

$A_s =$ 4 ϕ 12



* **Design of Columns.**

Take the column (350*1000)

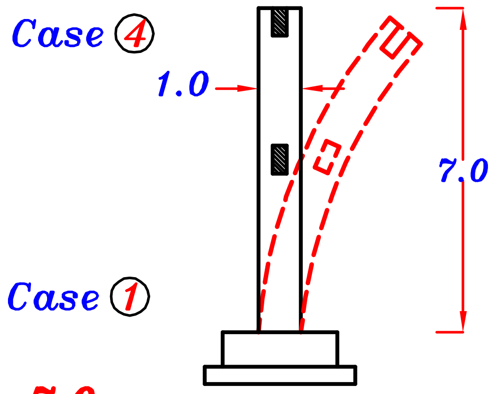
Design the critical Col.

at the Roller support

$$P = \frac{\Sigma F}{2} = 807.75 \ kN$$

Check Buckling.

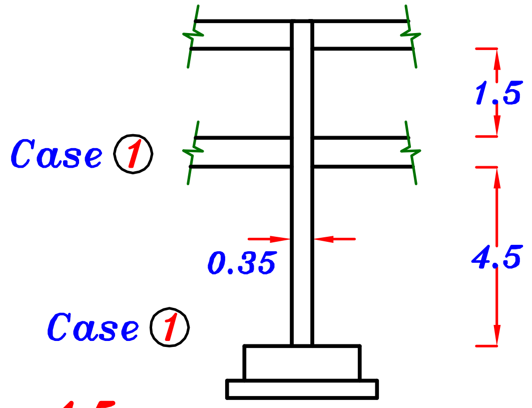
① In Plane.



$H_o = 7.0\ m$

$\lambda_b = \frac{K * H_o}{t} = \frac{2.2 * 7.0}{1.0} = 15.4 > 10$

② Out of Plane.



$H_o = 4.5\ m$

$\lambda_b = \frac{K * H_o}{b} = \frac{1.2 * 4.5}{0.35} = 15.4 > 10$

$$\delta = \frac{(\lambda_b)^2 * t}{2000} = \frac{15.4^2 * 1.0}{2000} = 0.12\ m$$

$$M_{add.} = P * \delta = 807.75 * 0.12 = 96.93\ \ kN.m$$

$$e = \frac{M}{P} = \frac{96.93}{807.75} = 0.12\ \ m \qquad \therefore \frac{e}{t} = \frac{0.12}{1.0} = 0.12 < 0.5 \xrightarrow{use} I.D.$$

$\zeta = \frac{1.0 - 0.1}{1.0} = 0.9$ $\xrightarrow{use}$ ECCS Design Aids Page 4–23

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\ b\ t} &= \frac{807.75 * 10^3}{25 * 350 * 1000} = 0.092 \\ \frac{M_U}{F_{cu}\ b\ t^2} &= \frac{96.93 \ * 10^6}{25 * 350 * 1000^2} = 0.011 \end{aligned}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$$As = As' = \mu * b * t = \rho * F_{cu} * 10^{-4} * b * t = 1.0 * 25 * 10^{-4} * 350 * 1000 = 875\ mm^2$$

$$As_{total} = As + As' = 1750\ \ mm^2$$

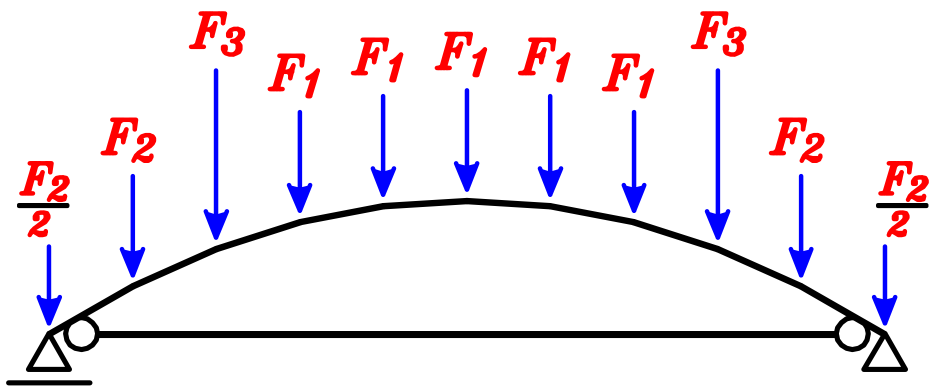
$$As_{min} = \frac{0.25 + 0.052\ \lambda_{max}}{100} * b * t$$

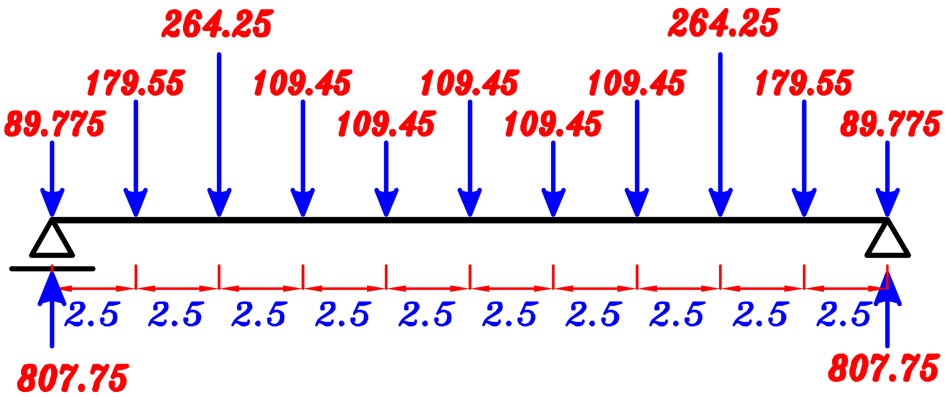
$$= \frac{0.25 + 0.052\ (15.40)}{100} * 350 * 1000 = 3677.8\ mm^2 < As_{total}\ \therefore\ O.K.$$

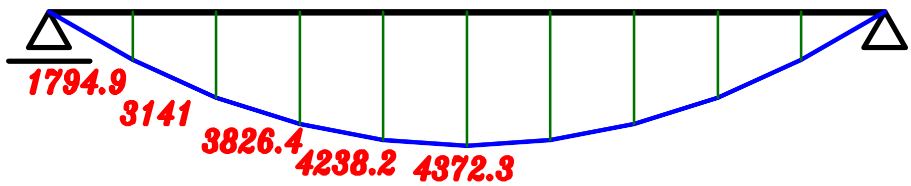
$As = As' = \ 1750\ \ mm^2$ 5 ϕ 22

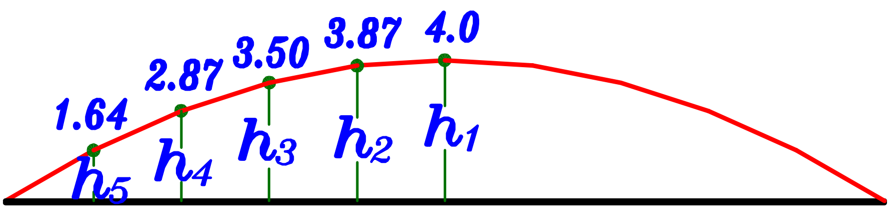
Drawing Arch Girder at axis 8-8.

عند رسم ال **Arch Girder** عند **axis 8-8** الاحمال عليه غير متساويه لذا عند
تحديد ارتفاعاته لن ينفع أن نضع أحمال **1 kN** عند كل **Joint**
لذا يجب أن نرسم ال **moment** الحقيقى أولا ثم نحدد الارتفاعات
على أساس أن نفس النسب بين الارتفاعات هى نفس النسب بين العزوم الحقيقيه









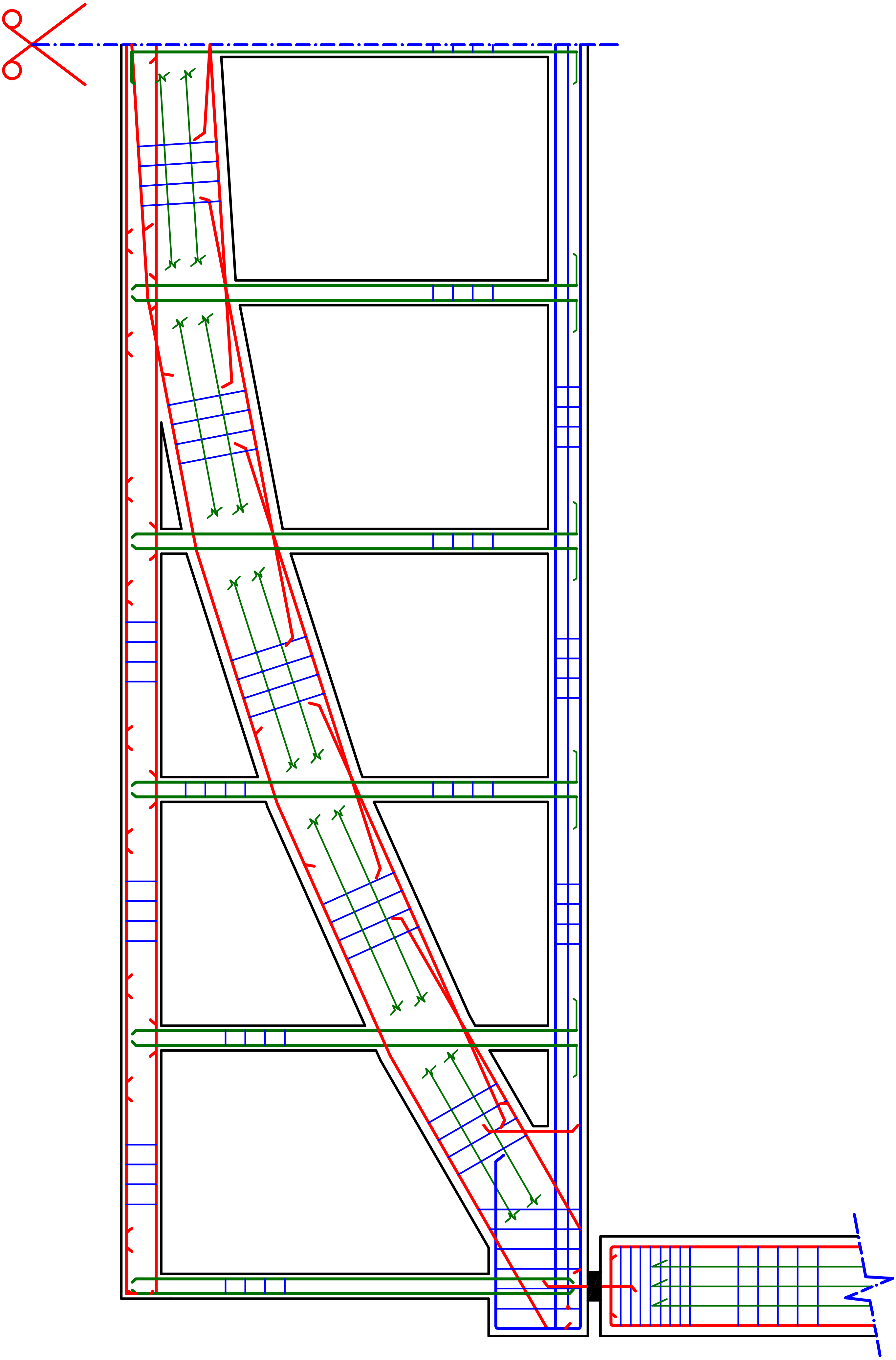
$M_1 = 4372.3$ kN.m $\longrightarrow$ $h_1 = 4.0$ m

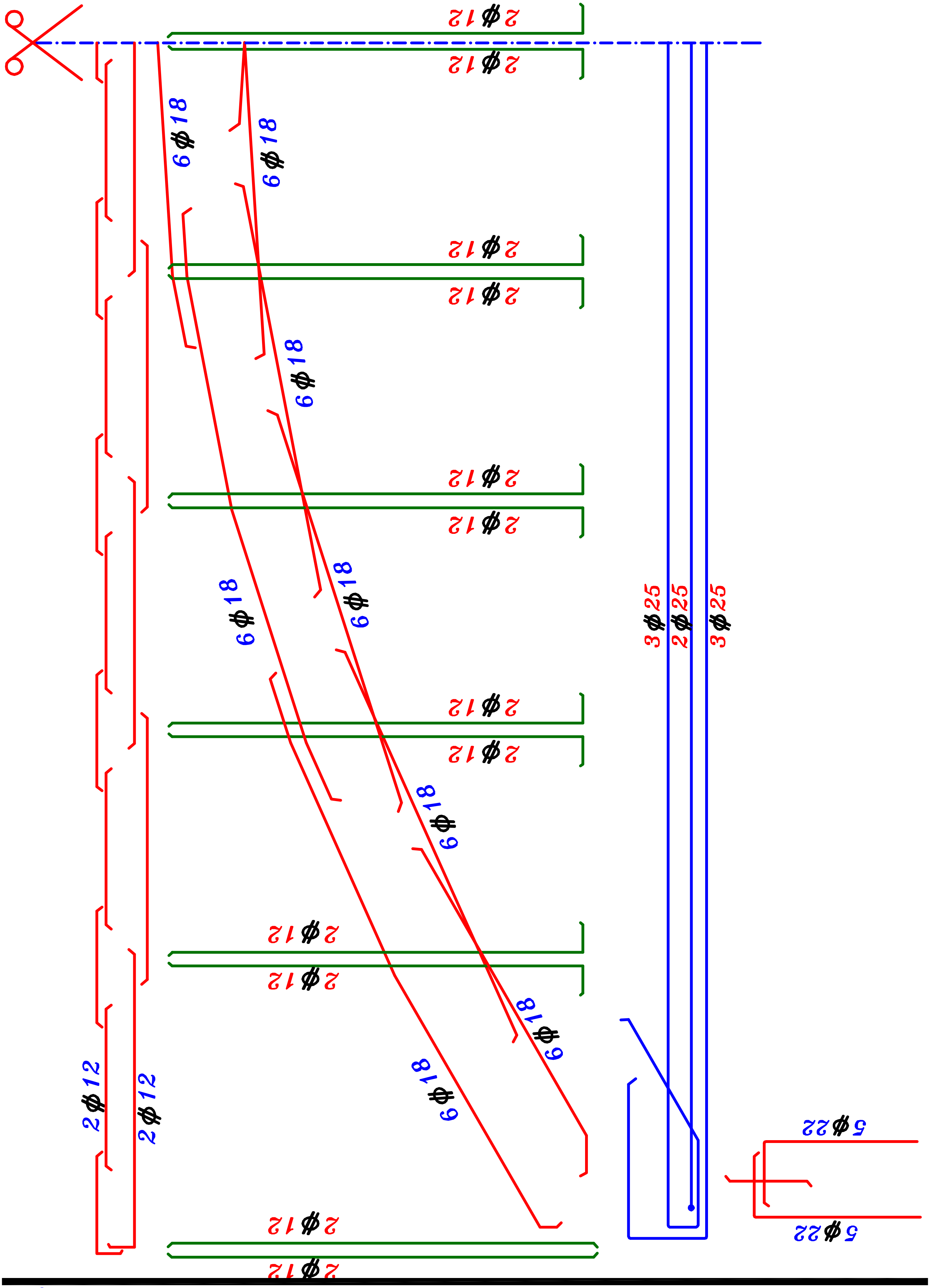
$M_2 = 4238.2$ kN.m $\longrightarrow$ $h_2 = 3.87$ m

$M_3 = 3826.4$ kN.m $\longrightarrow$ $h_3 = 3.50$ m

$M_4 = 3141$ kN.m $\longrightarrow$ $h_4 = 2.87$ m

$M_5 = 1794.9$ kN.m $\longrightarrow$ $h_5 = 1.64$ m

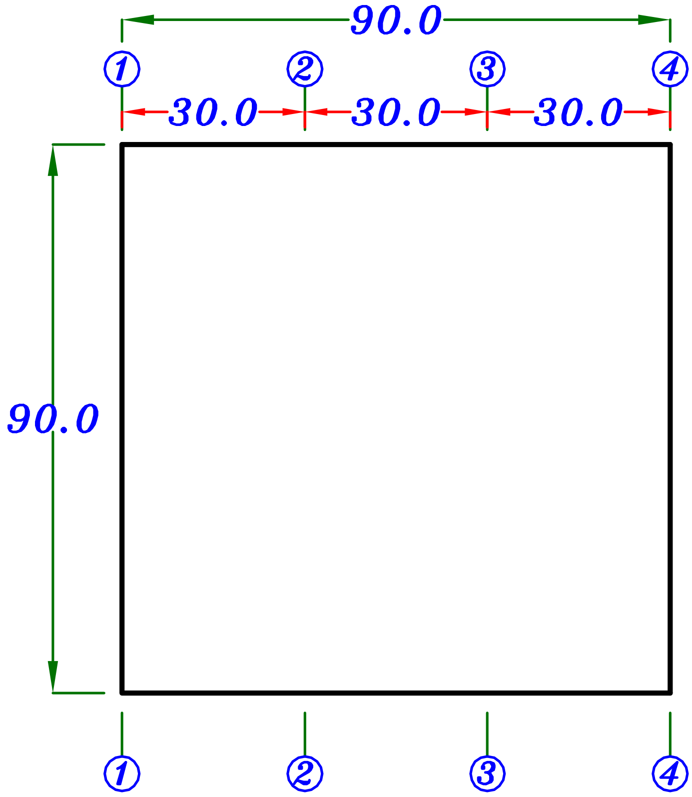



2 ϕ 12
6 ϕ 18
3 ϕ 25
2 ϕ 25
3 ϕ 25
5 ϕ 22
5 ϕ 22

Example.

Choose a convient system
It is allowed to put columns
only at axes 1, 2, 3 & 4
Draw Plan and elevation.

Weak Soil.

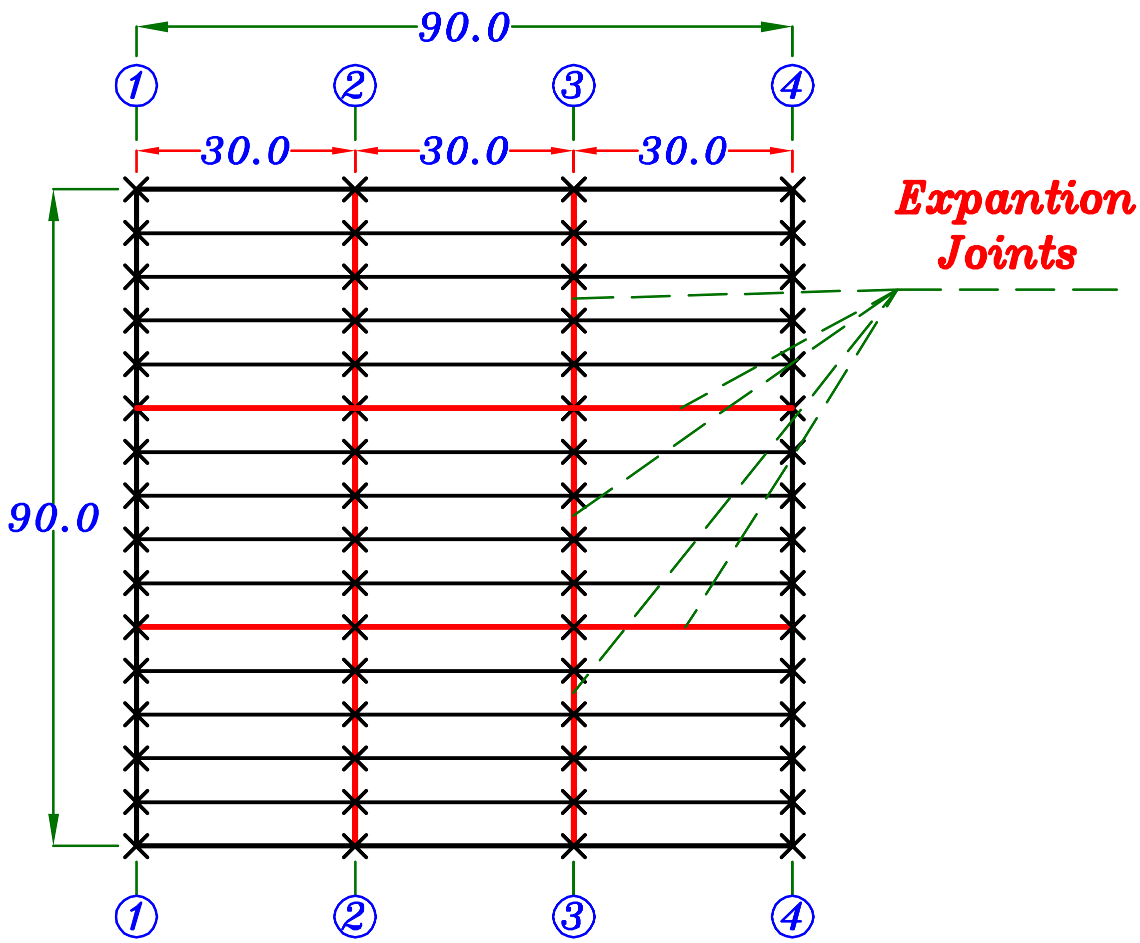


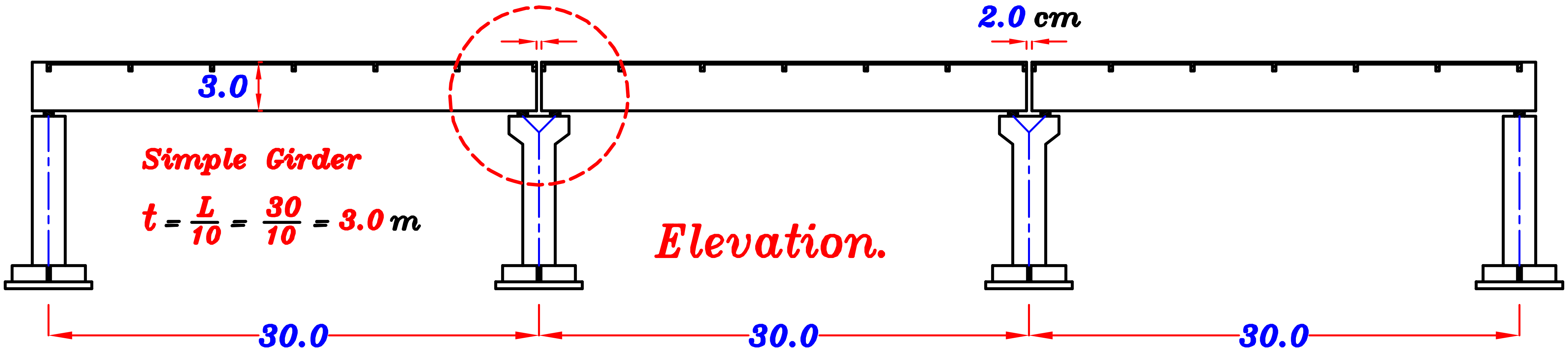
يتم عمل فواصل تمدد فى الاتجاهين كل ٣٠ م

لان التربه ضعيفه فيجب أختيار **Determinate system**

فنختار **simple girder** محمول على **Real Hinge and Roller**

Key Plan

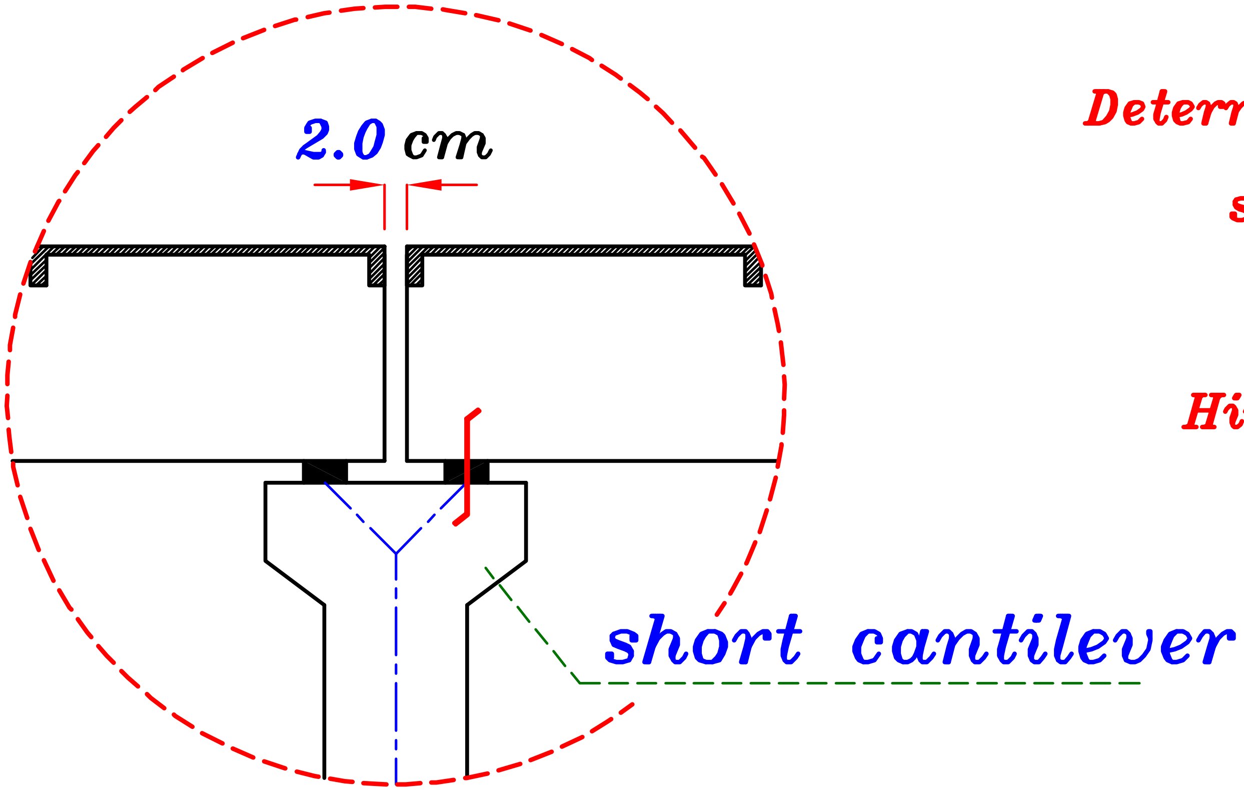




Simple Girder

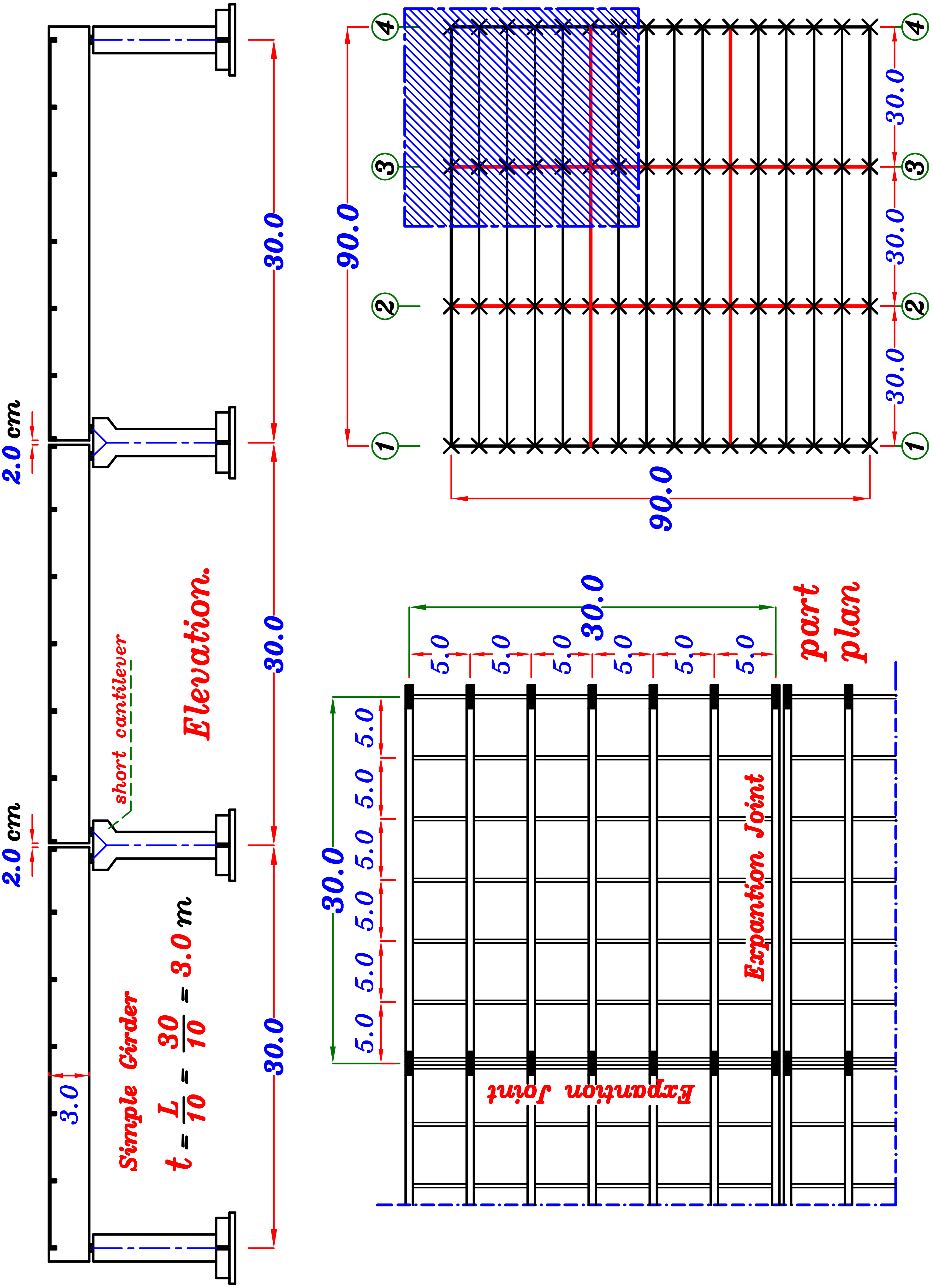
$$t = \frac{L}{10} = \frac{30}{10} = 3.0\,m$$

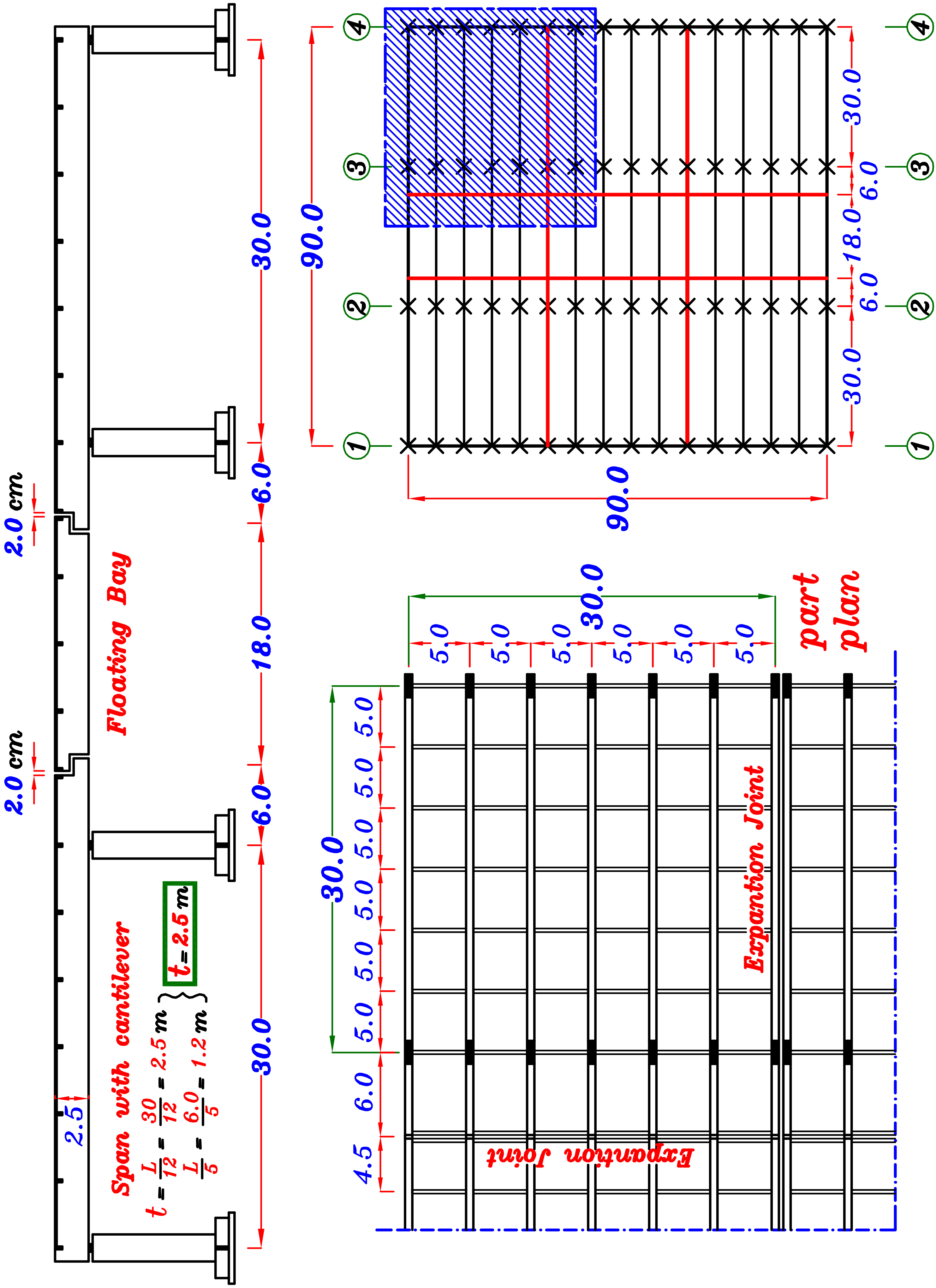
Elevation.



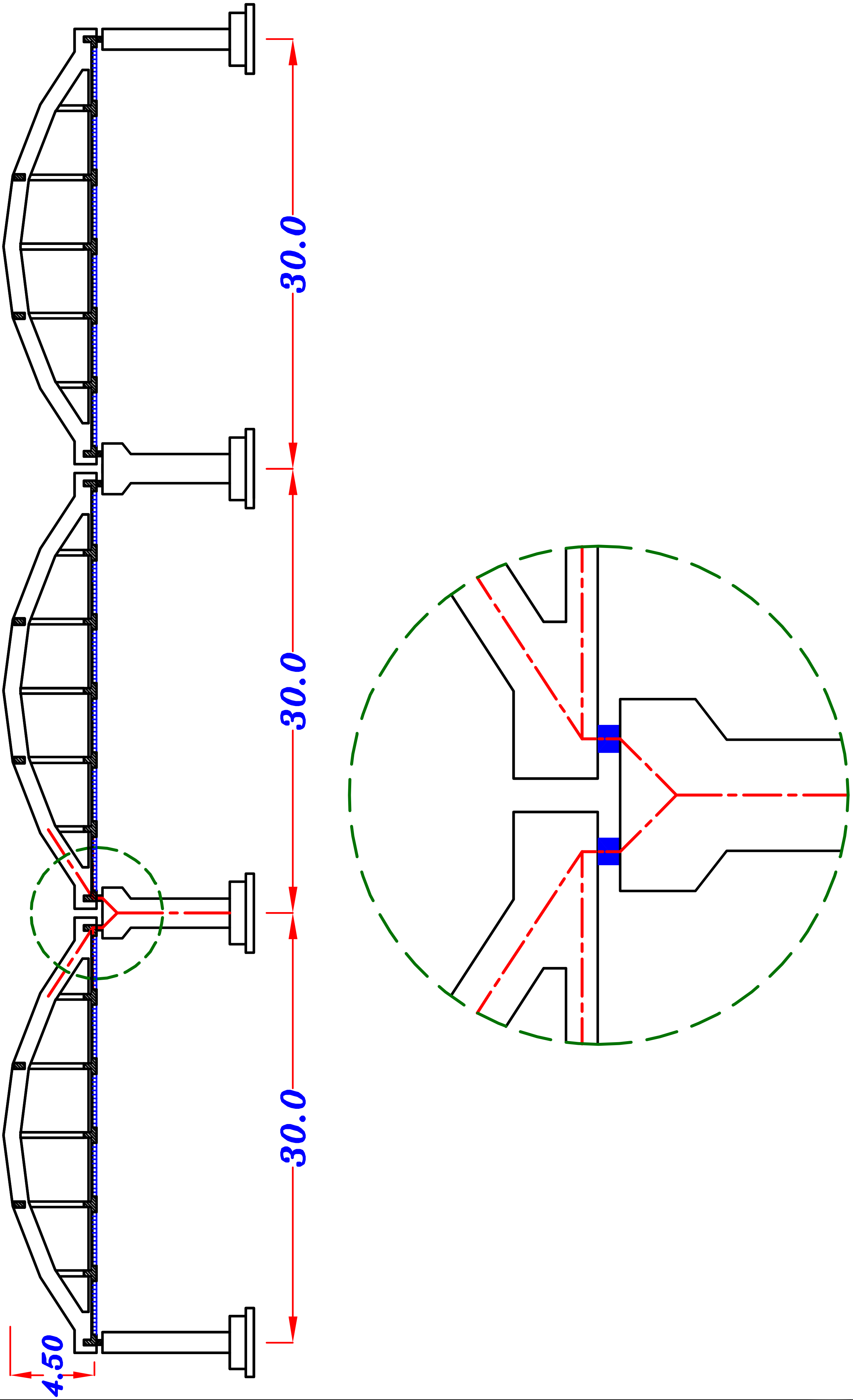
Determinate Girder لكى يكون

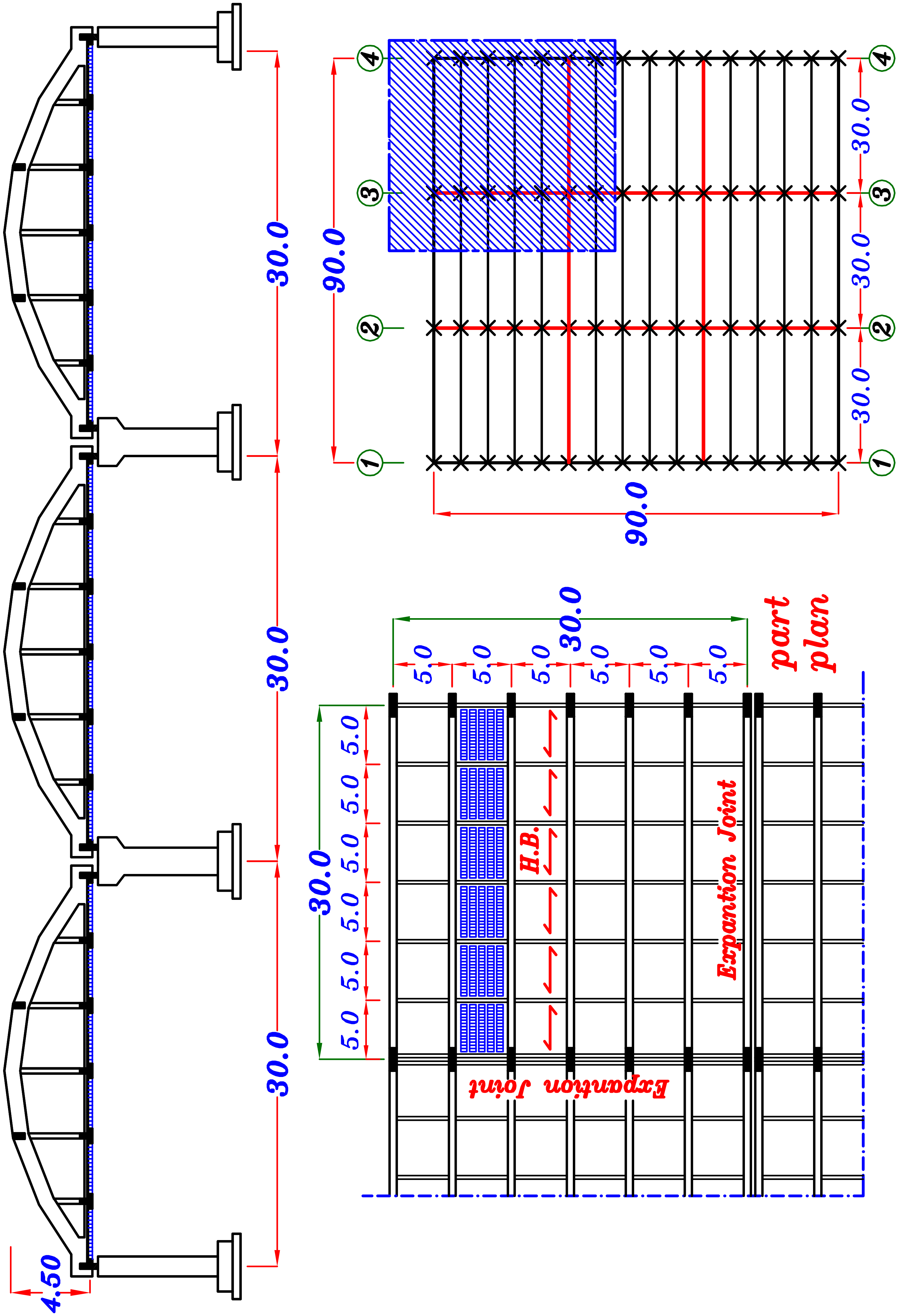
supports يجب أن يكون ال


Elevation.
Simple Girder
t = L/10 = 30/10 = 3.0 m
short cantilever
2.0 cm
2.0 cm
3.0
30.0
30.0
30.0
90.0
90.0
30.0
30.0
30.0
part plan
30.0
30.0
5.0
Expantion Joint
Expantion Joint


2.0 cm
2.0 cm
Floating Bay
Span with cantilever
2.5
t = L/12 = 30/12 = 2.5 m
L/5 = 6.0/5 = 1.2 m
t = 2.5 m
30.0
6.0
18.0
6.0
30.0
90.0
90.0
30.0
6.0
18.0
6.0
30.0
part plan
30.0
5.0 5.0 5.0 5.0 5.0 5.0
30.0
5.0 5.0 5.0 5.0 5.0 5.0
6.0
4.5
Expantion Joint
Expantion Joint

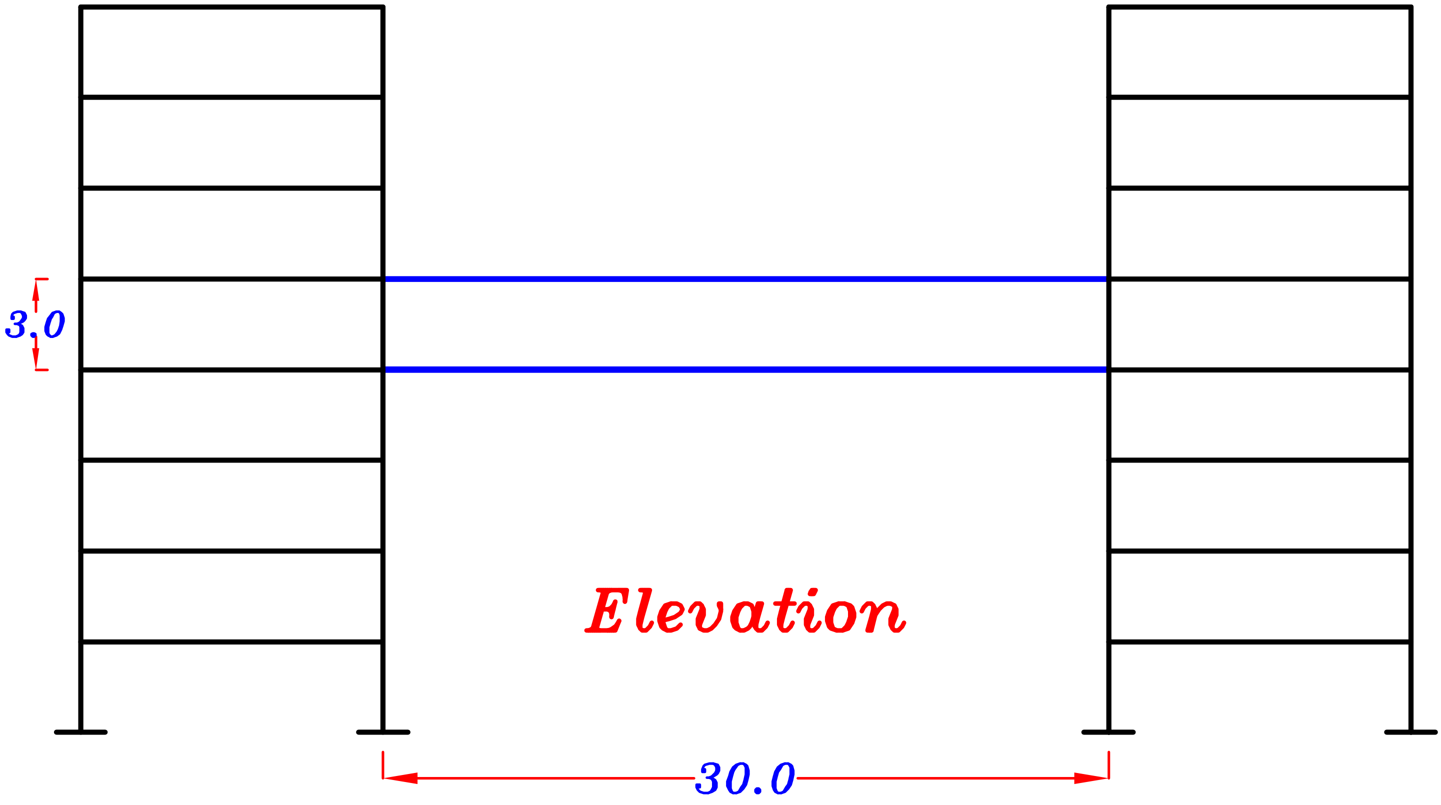
Better Solution.

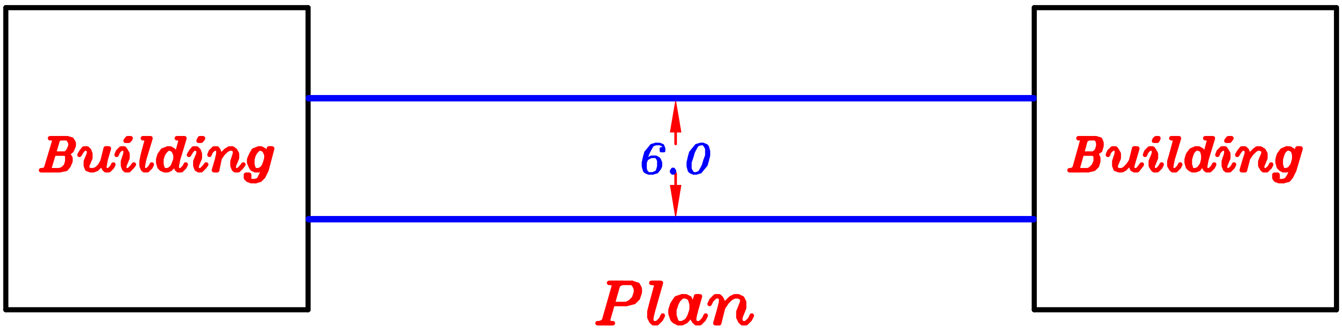




30.0
30.0
30.0
4.50
90.0
90.0
1
2
3
4
part plan
30.0
5.0
H.B.
Expantion Joint
Expantion Joint

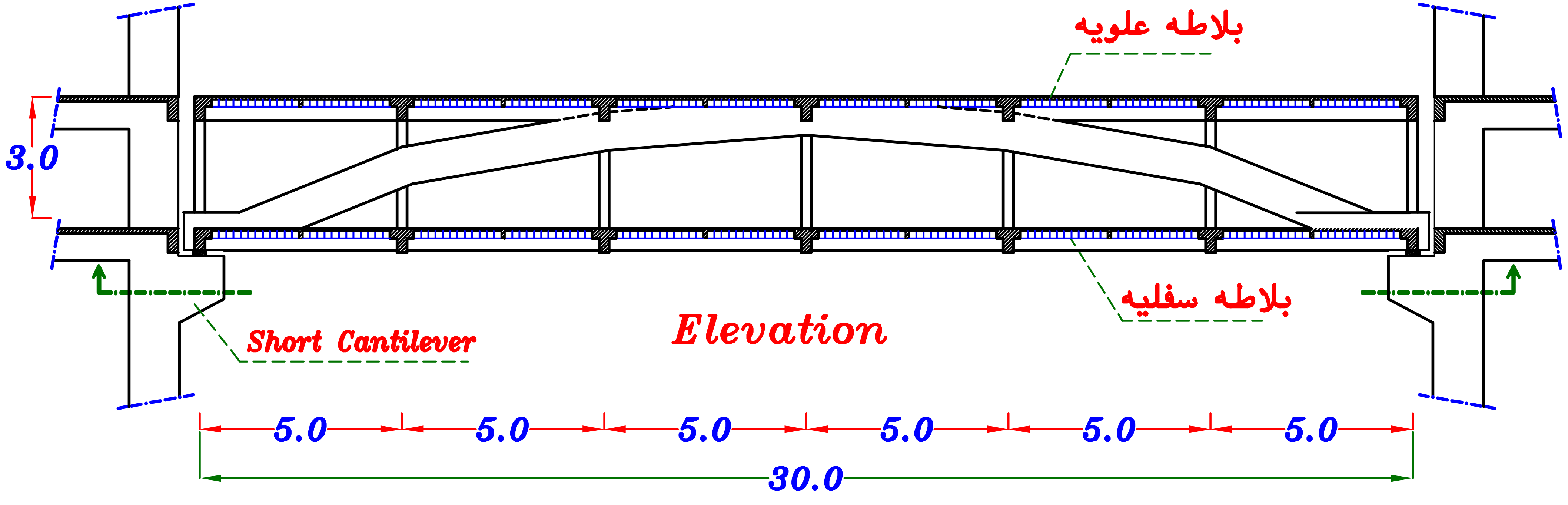
Example.

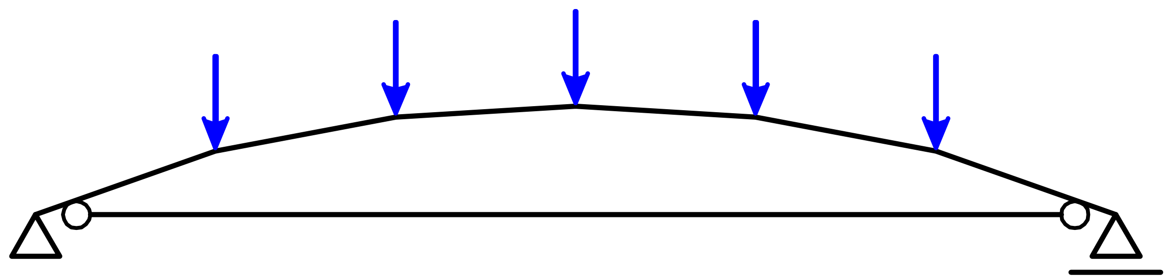




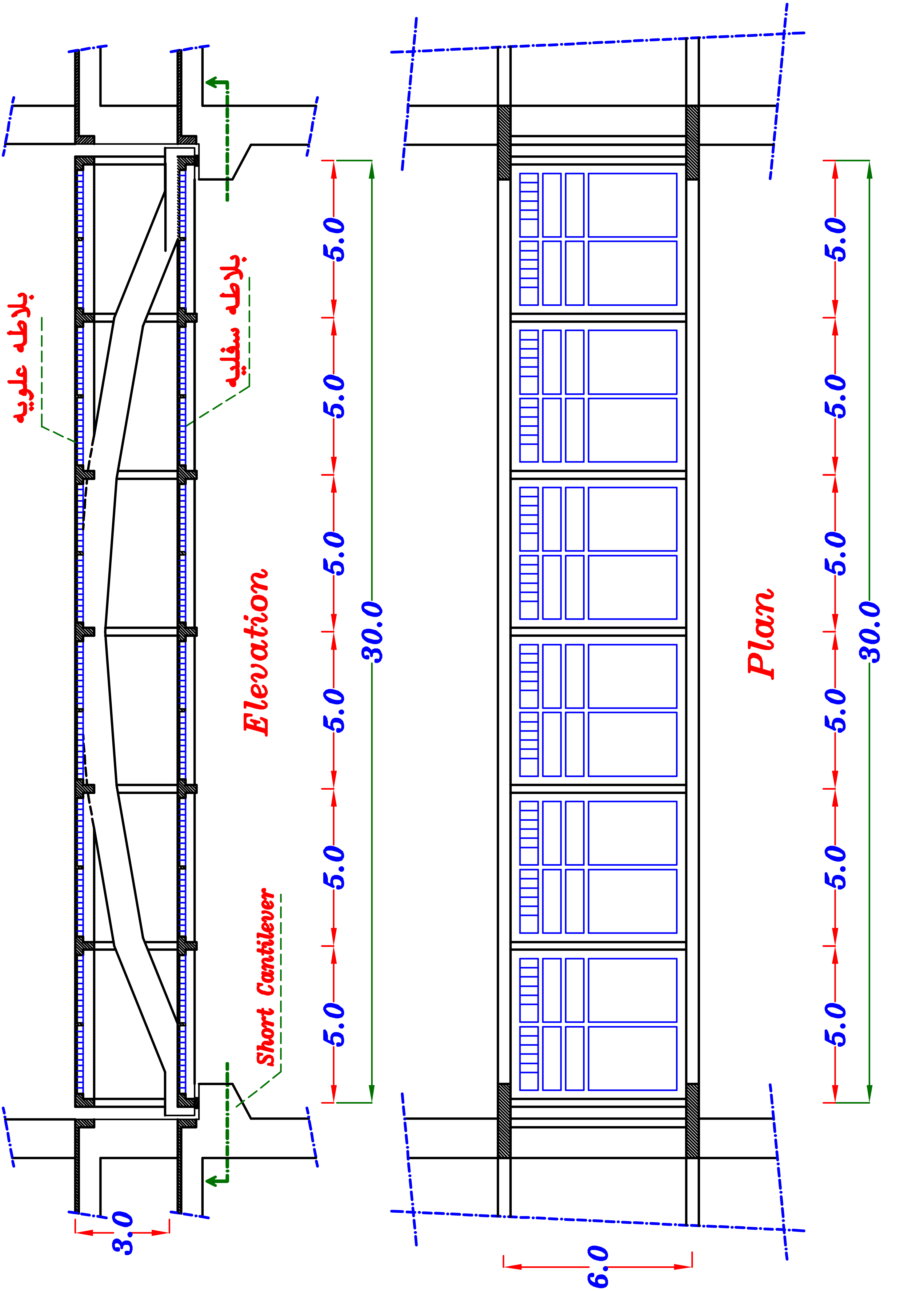
IF we want to construct a walk bridge between two buildings at the Fourth Floor.

Choose a statical system and draw concrete dimensions For the system in elevation , Plan & Side View.

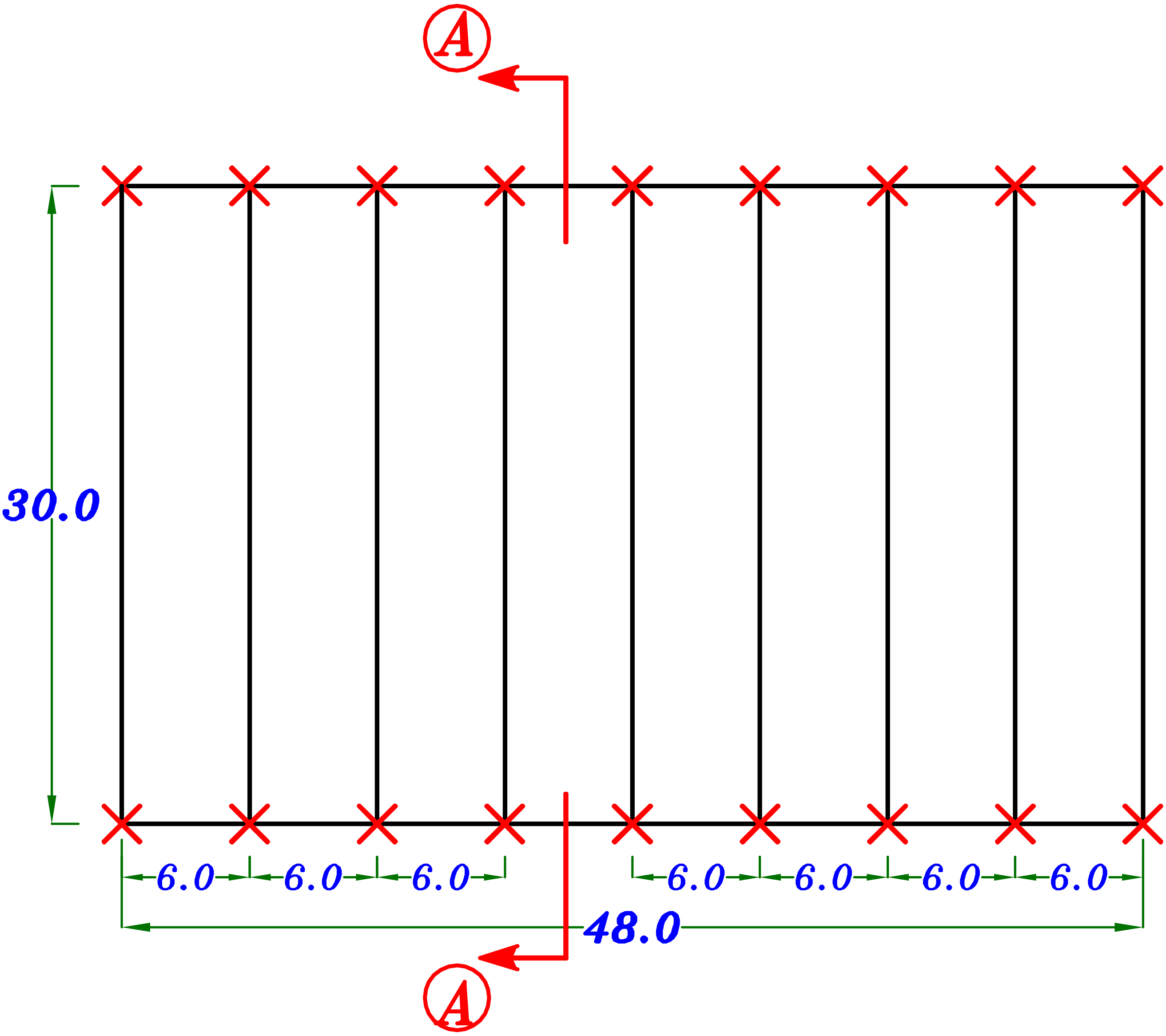


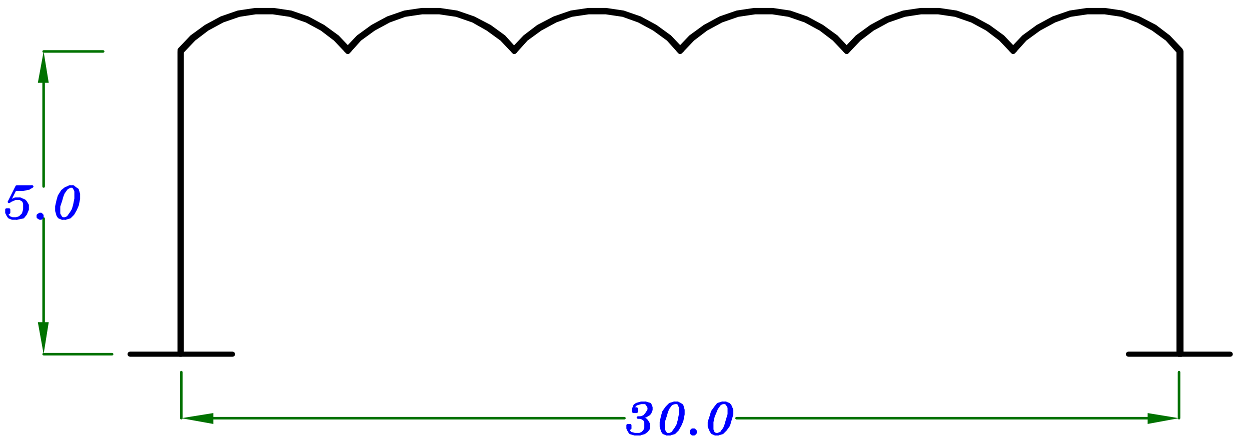


يوجد فى كُبرى المشاه بلاطه سفليه للسير عليها و بلاطه علويه كتغطيه للكُبرى
و ممكن عمل بلاطه سفليه فقط.

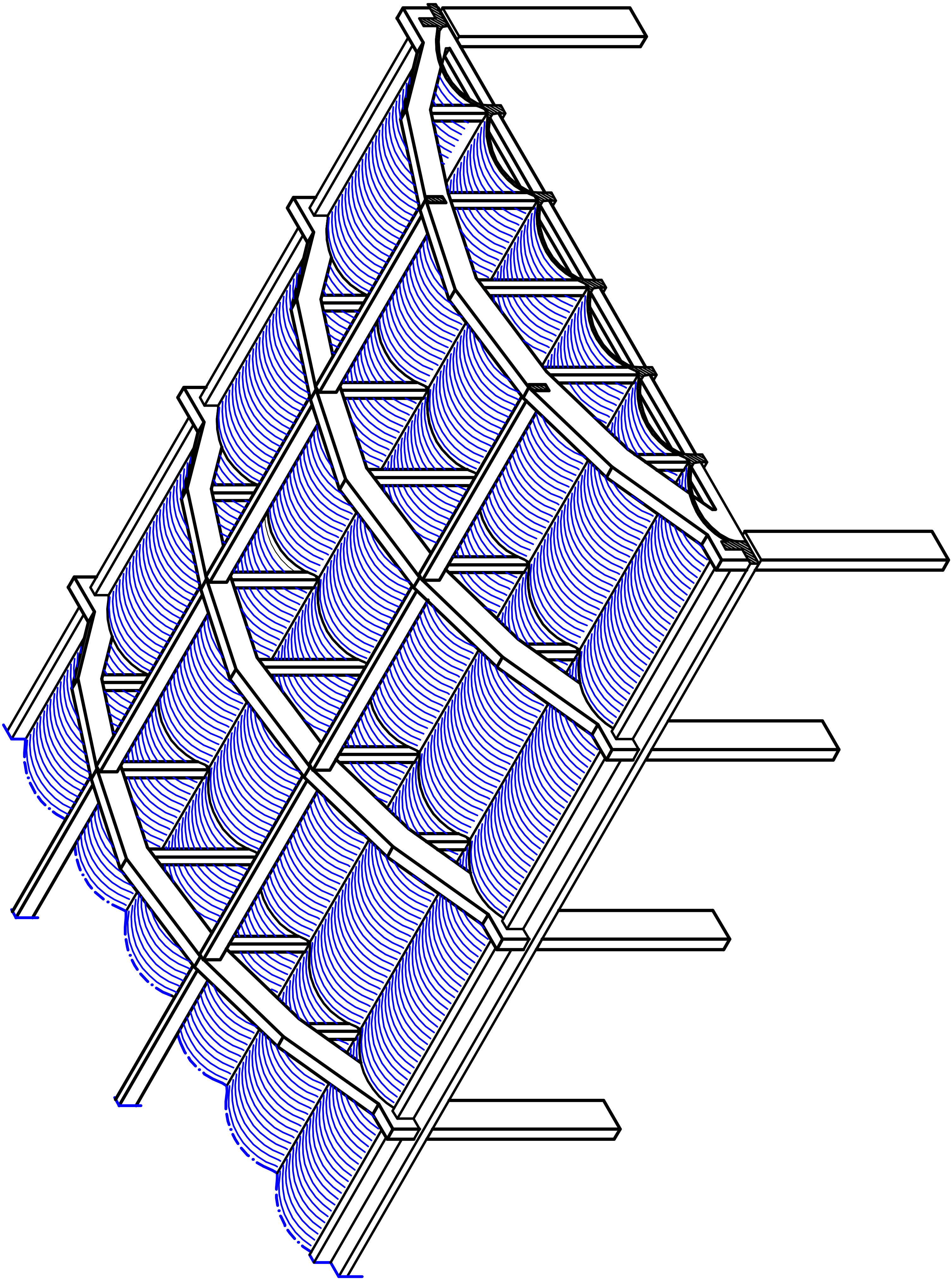

بلاطه علويه
بلاطه سفليه
Elevation
Short Cantilever
5.0
5.0
5.0
5.0
5.0
5.0
30.0
3.0
Plan
5.0
5.0
5.0
5.0
5.0
5.0
30.0
6.0

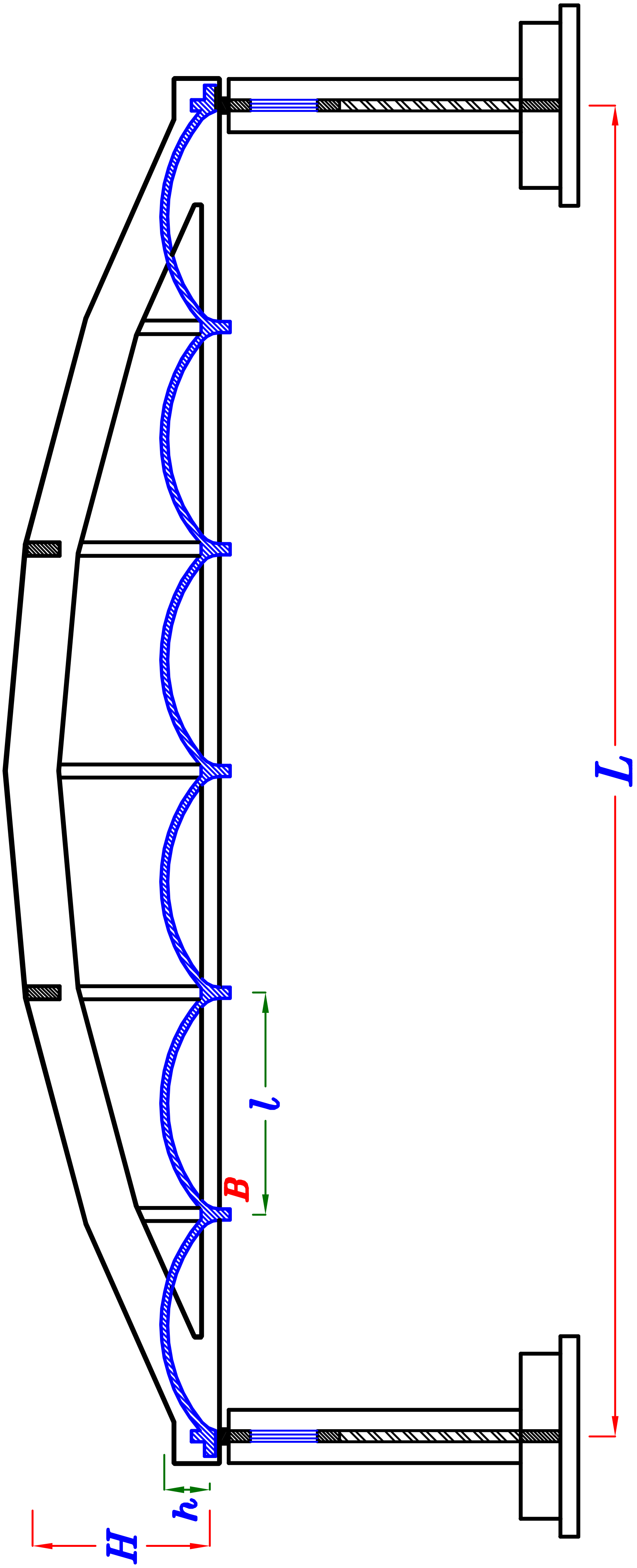
Example.



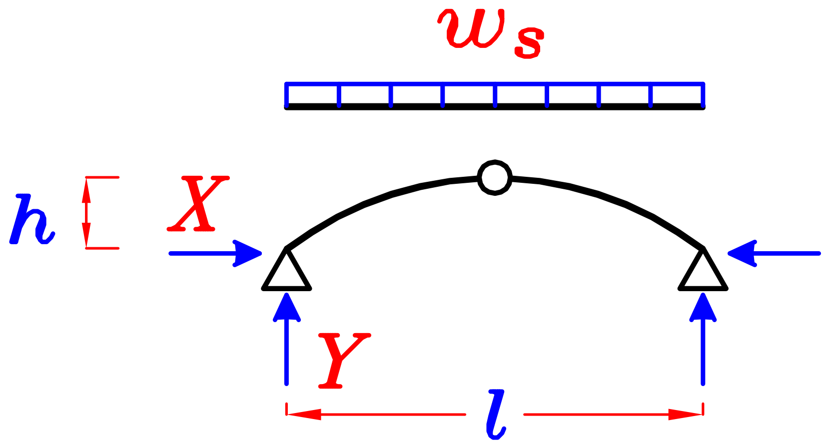


Sec. (A–A)




L
l
B
h
H

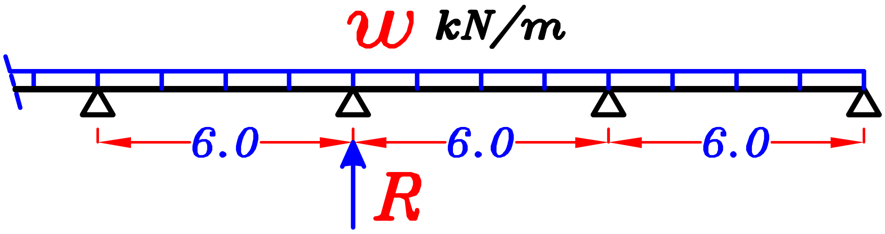
Arch slab



$$Y = \frac{w_s \, l}{2}$$

$$X = \frac{w_s \, l^2}{8 \, h}$$

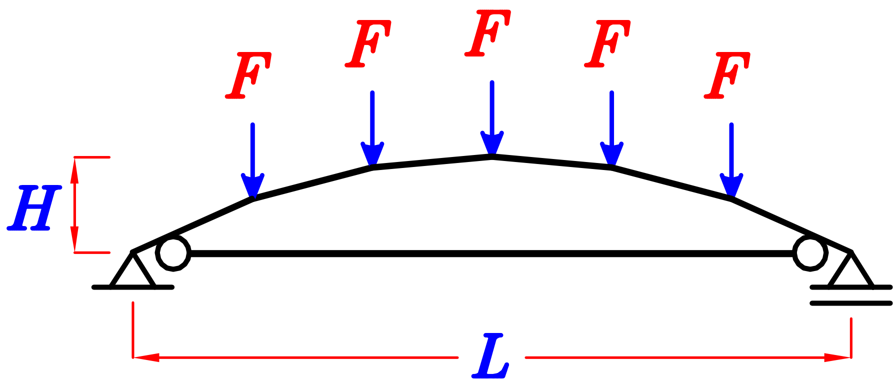
Beam B



$$w = o.w. + 2\,Y$$

$$R = w * S$$

Arch Girder

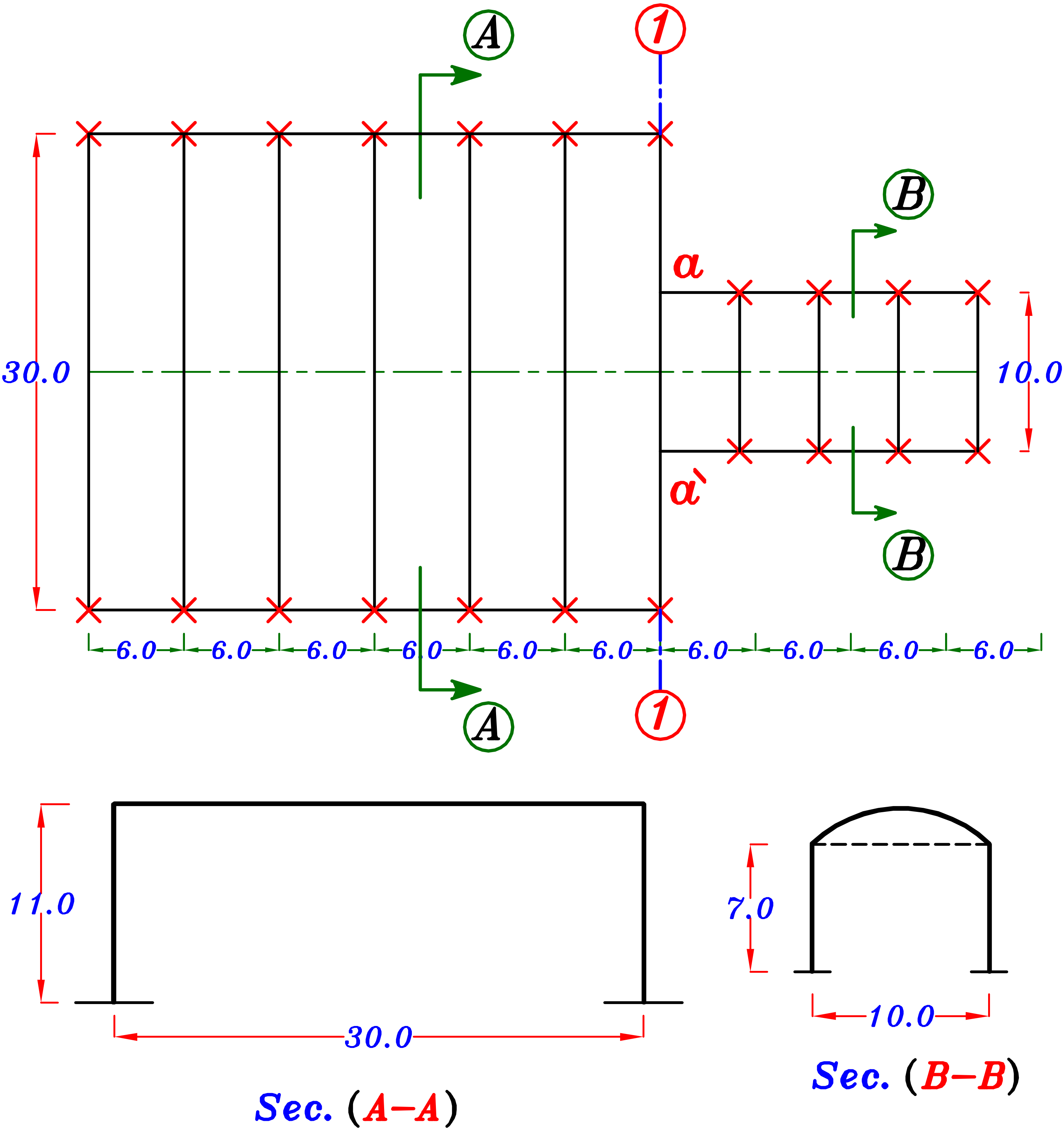


$$F = R + O.W.(\text{Arch Girder}) * \alpha$$

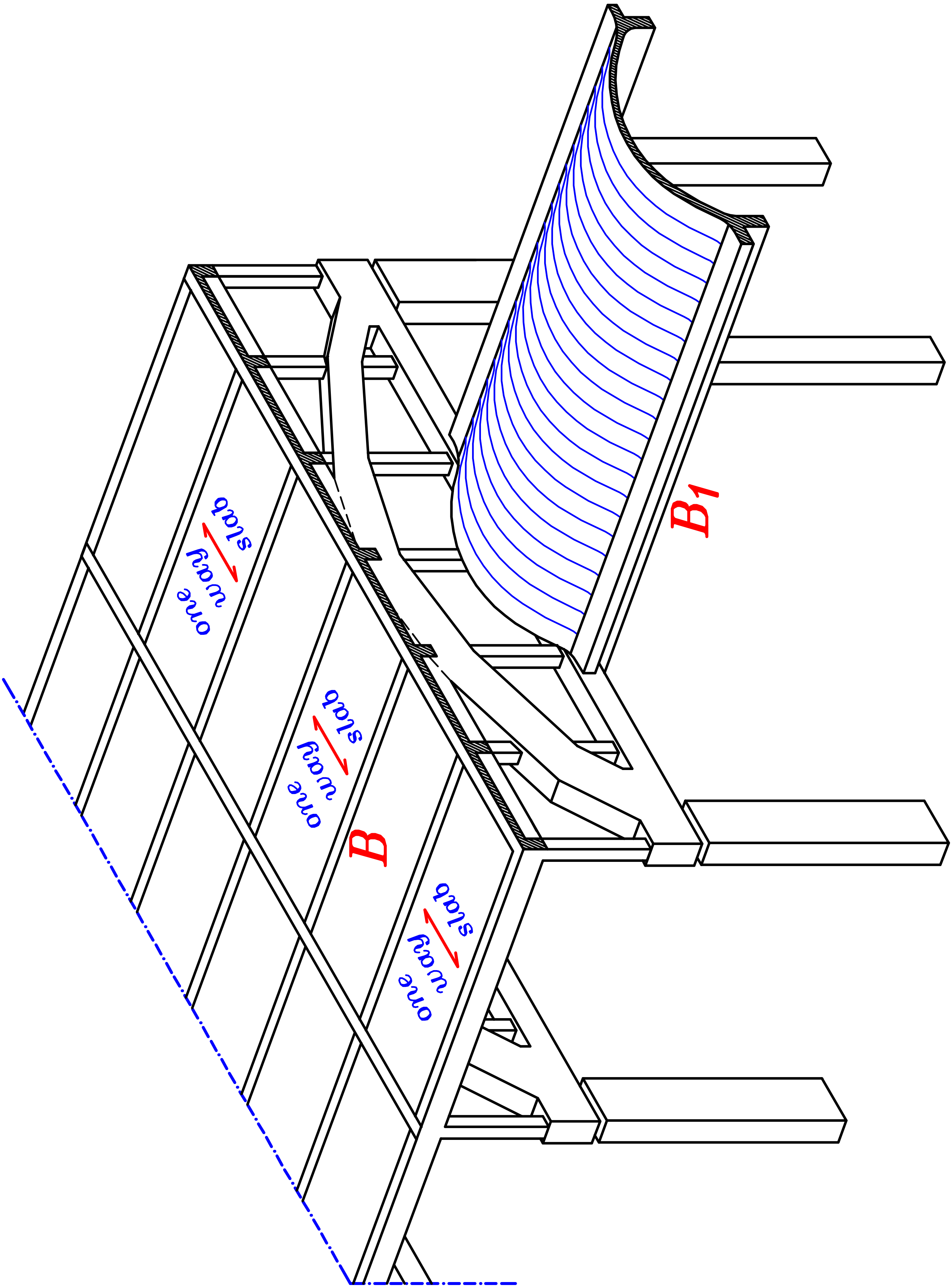
$$Tie = T(\text{Arch Girder}) + T(\text{Arch slab})$$

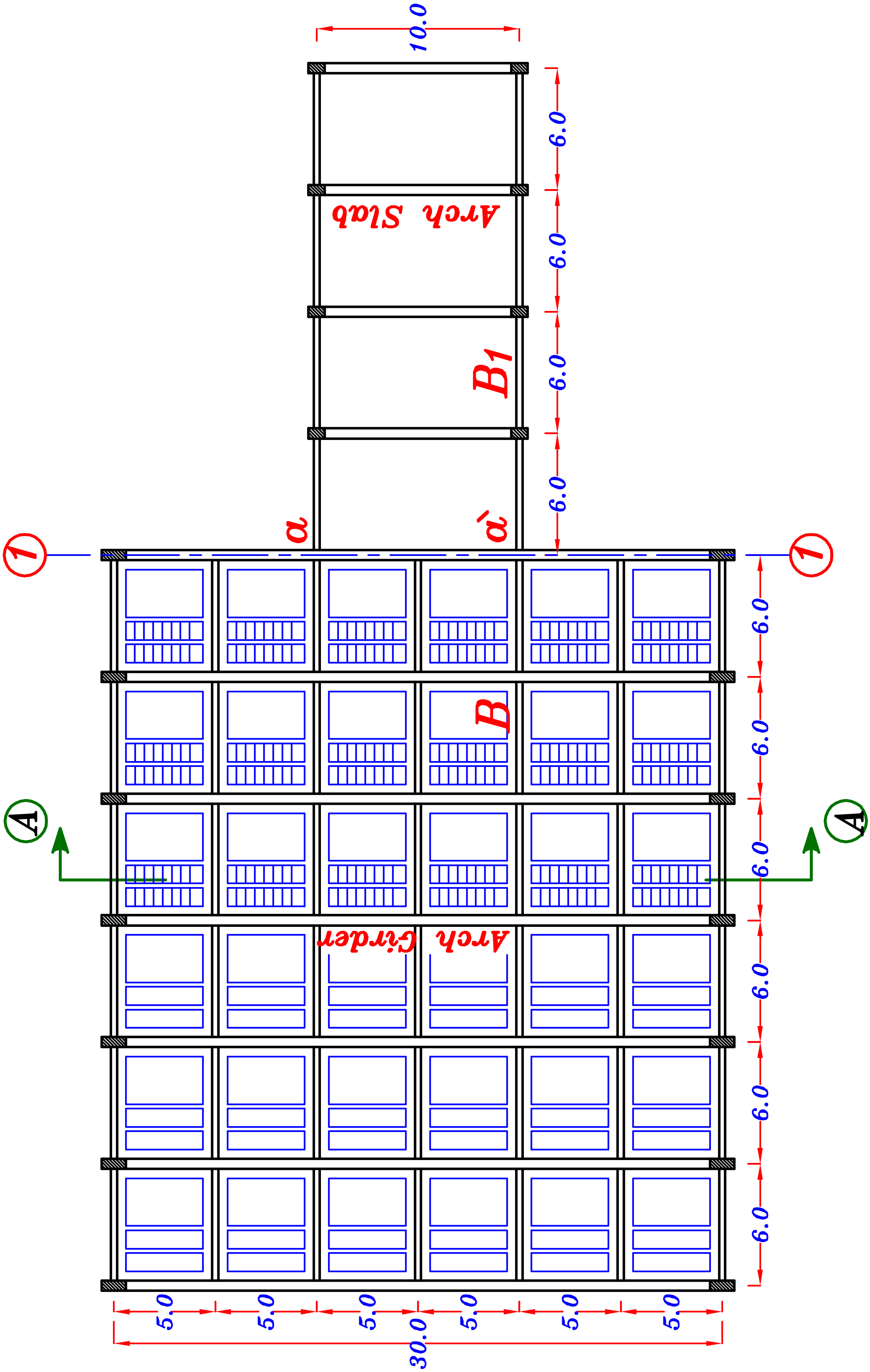
$$= 0.95 \, \frac{M_o}{H} + X * S$$

Example.



Design the system at axis 1


one way slab
one way slab
one way slab
B
B1

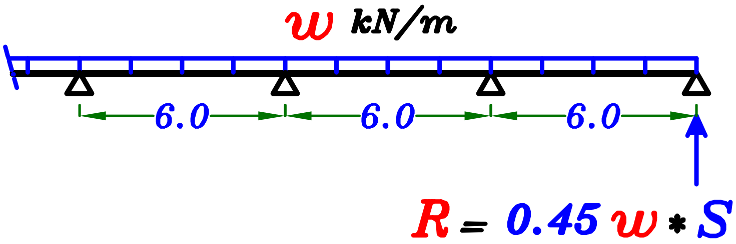


10.0
Arch Slab
B1
a
a`
1
1
B
A
A
Arch Girder
6.0
5.0
30.0

Beam B

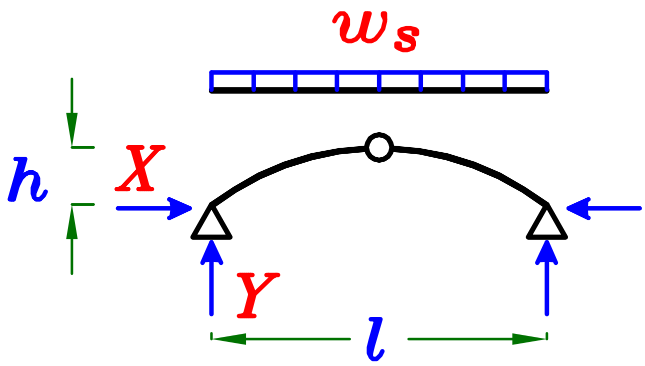
$$w = o.w. + \left(\frac{w_{rib}}{S}\right) * a$$

$$R = 0.45\, w * S$$



Arch slab

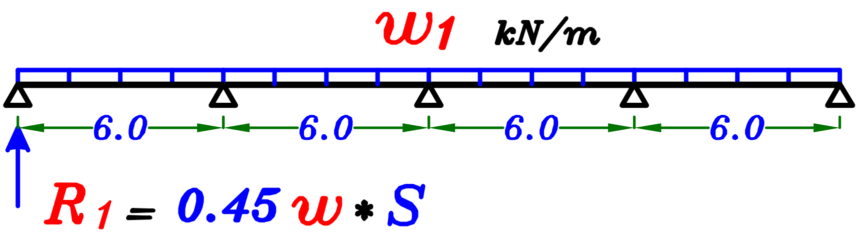
$$Y = \frac{w_s\, l}{2}\ , \quad X = \frac{w_s\, l^2}{8\, h}$$



Beam B_1

$$w_1 = o.w. + Y$$

$$R_1 = 0.45\, w_1 * S$$



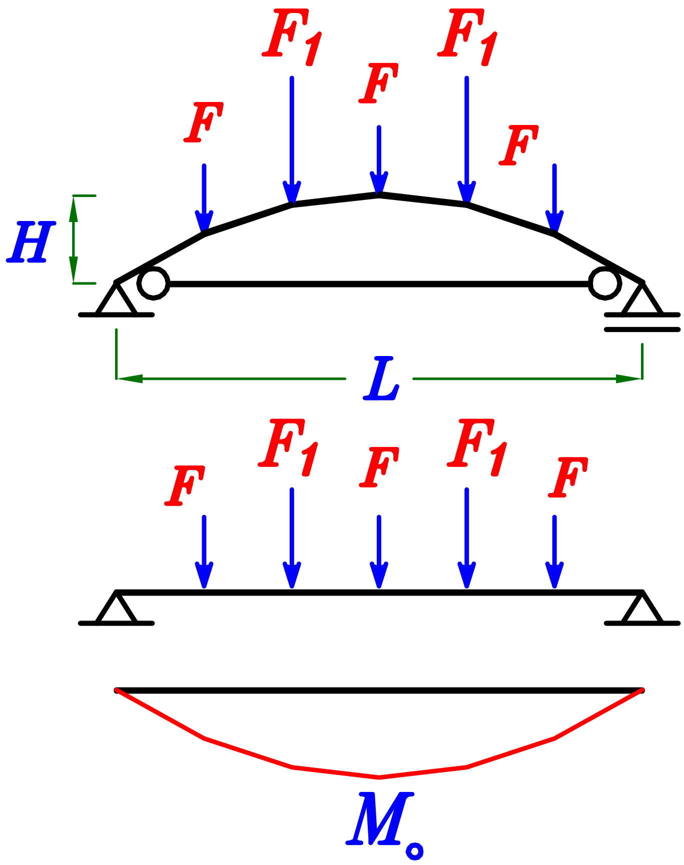
Arch Girder at axis 1–1

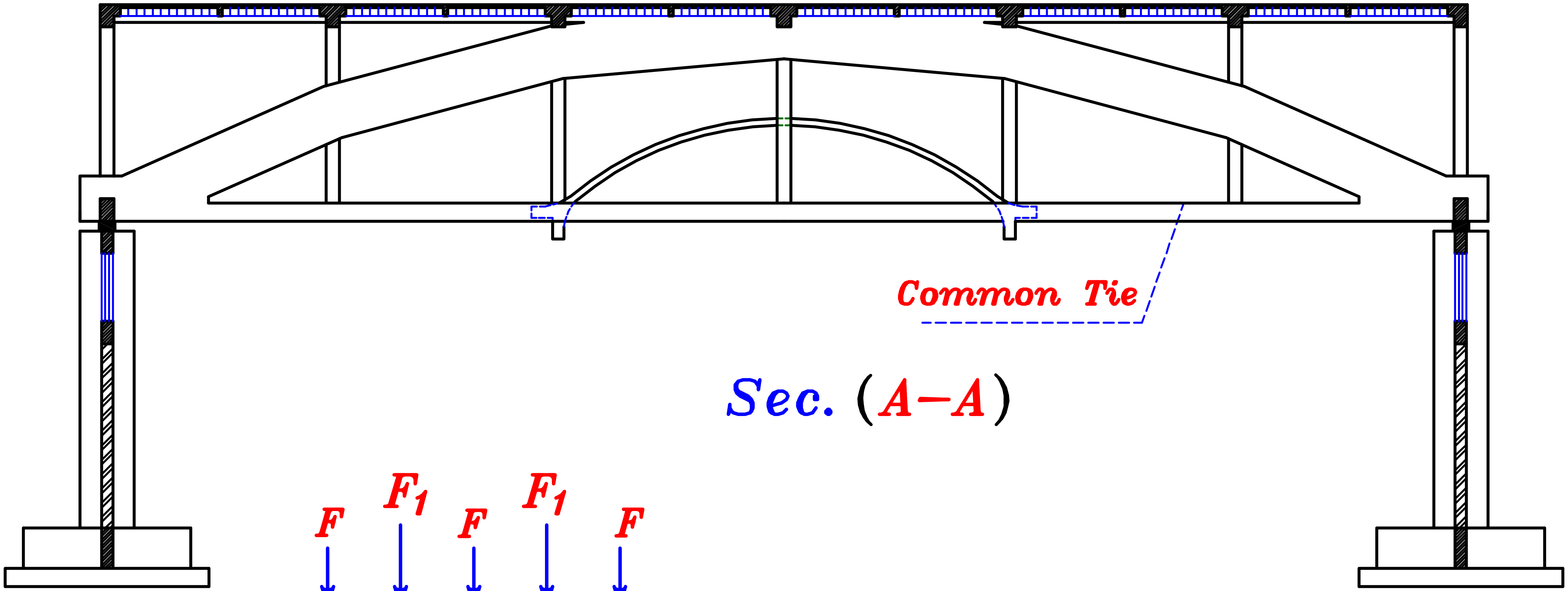
$$F = R + O.W. * a$$

$$F_1 = R + R_1 + O.W. * a$$

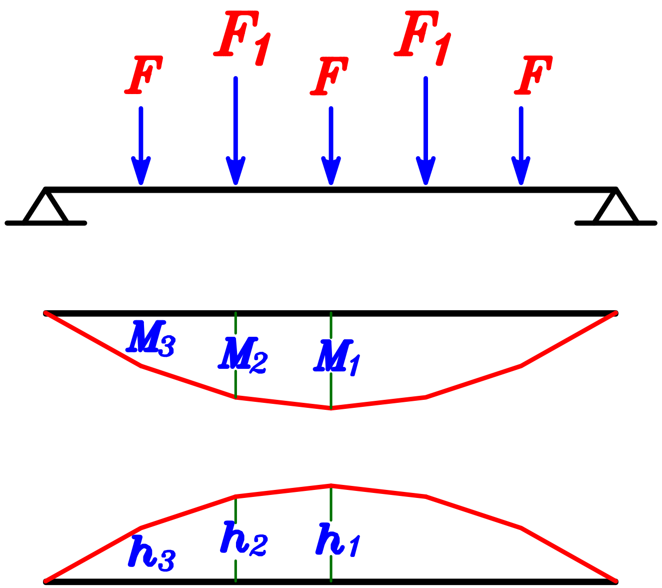
$$Tie = T(\text{Arch Girder}) + T(\text{Arch slab})$$

$$= 0.95\, \frac{M_o}{H} + X * S$$





Sec. (A–A)



Drawing Arch Girder at axis 1–1

عند رسم ال Arch Girder عند axis 1–1 الاحمال عليه غير متساويه لذا عند تحديد ارتفاعاته لن ينفع أن نضع أحمال 1 kN عند كل Joint لذا يجب أن نرسم ال moment الحقيقى أولا ثم نحدد الارتفاعات على أساس أن نفس النسب بين الارتفاعات هى نفس النسب بين العزوم الحقيقيه

Reinforcement splices in Tie.

و صلات التسليح فى ال Tie .

اذا زاد طول السيخ عن ١٢,-م المفروض أن نعمل وصله فى سيخ الحديد .

و فى ال Tie يجب أن يكون نوع الوصله باللحام أو وصله ميكانيكيه.

Welded or *Mechanical* splices.

أى لن ينفع معها وصلات بالتراكب *Lap* splices

Mechanical splices. الوصلات الميكانيكيه

يجب أن لا يقل قطر السيخ عن ١٦ مم $\boxed{min\ \phi\ 16}$

و يستخدم معها جلب من الحديد الصلب مواصفاته لا تقل عن مواصفات الاسياخ الموصوله

كما يجب أن لا تقل مقاومه قطاع الجلبه عن ١,٢٥ مره لـ F_y للاسياخ الموصوله .

و الوصله الميكانيكيه لها طريقتين للتنفيذ :

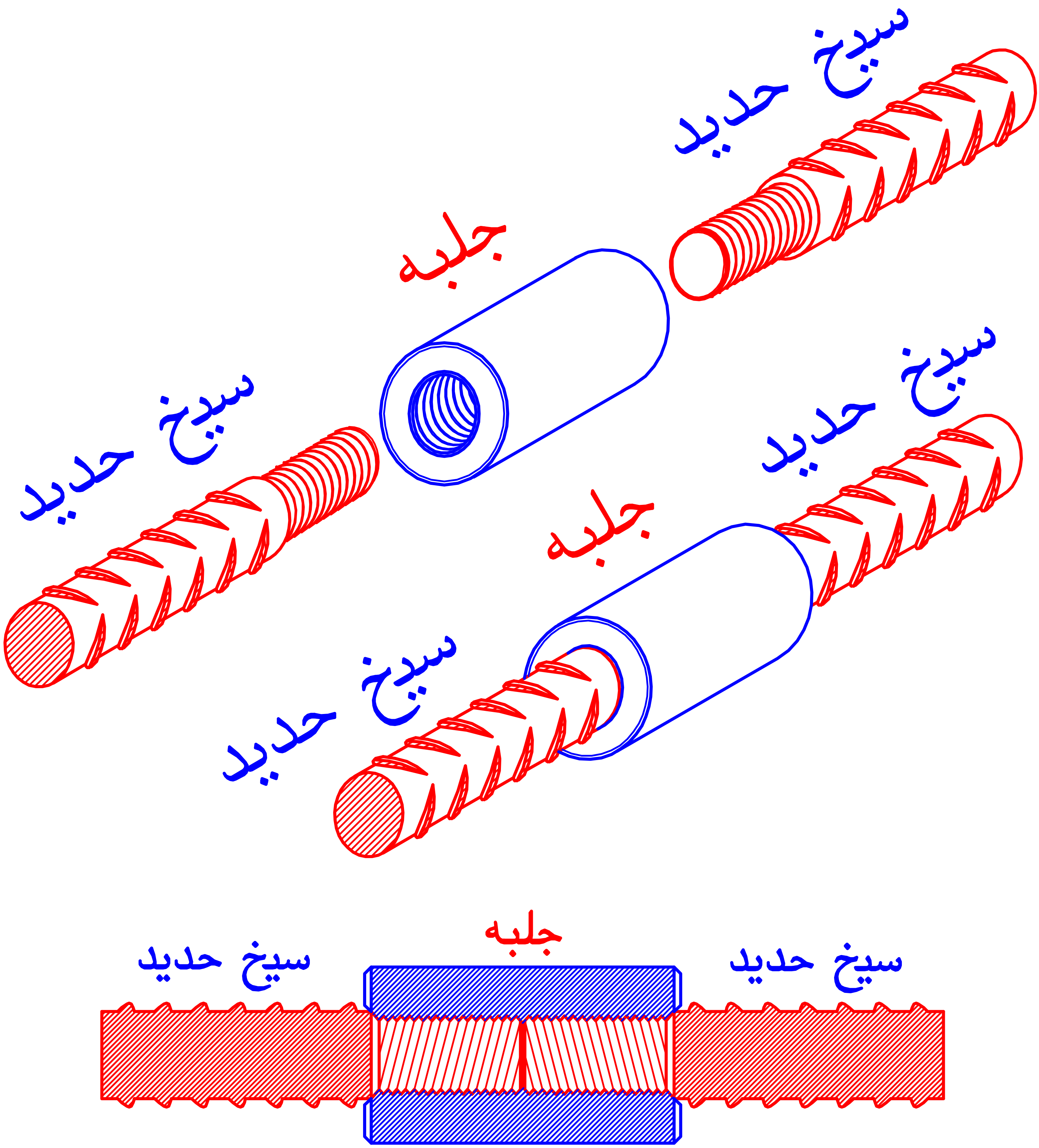
١ - بقلوظه الاسياخ من الخارج و الجلب من الداخل .

٢ - بضغط الجلب فى مكابس خاصه على نهايات الاسياخ ذات النتؤات .

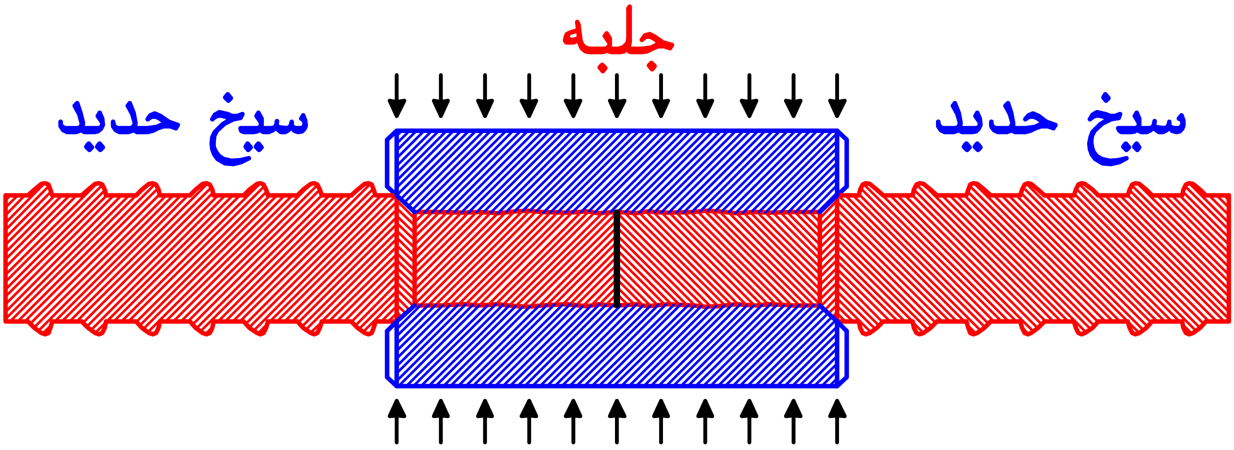
و الوصله الميكانيكيه لها طريقتين للتنفيذ :

١ ـ بقلوظه الاسياخ من الخارج و الجلب من الداخل .

تنتقل الاجهادات بين الاسياخ بواسطه الارتكاز بين اسنان قلوظ السيخ و اسنان قلوظ الجلبه .



٢- بضغط الجلب فى مكابس خاصه على نهايات الاسياخ ذات النتؤات لتنقل الاجهادات بين الاسياخ بواسطه الاحتكاك بين السطح الداخلى للجلبه مع السطح الخارجى لنهايه الاسياخ .



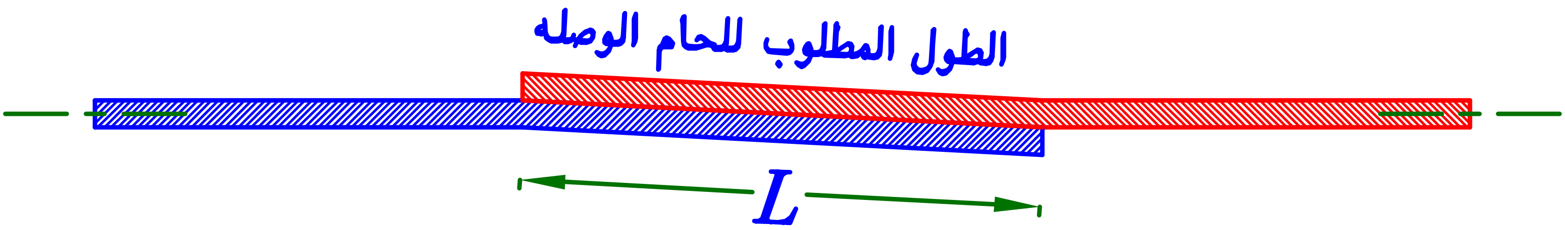
وصلات اللحام — *Welded splices.*

يجب أن لا يقل قطر السيخ عن ١٦ مم $\boxed{min\ \phi\ 16}$

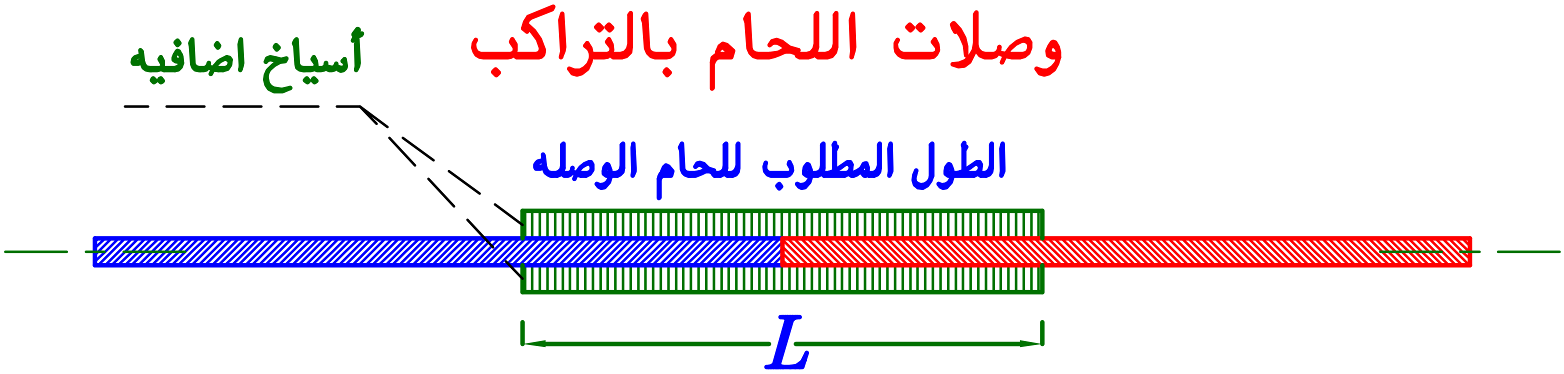
١- يستخدم لحام كهربائى .

٢- يجب أن يكون محور السيخين الملحومين على استقامه واحده .

٣- يجب ان لا تزيد مساحه الاسياخ الملحومه فى قطاع واحد عن ٢٥ ٪ و باقى الوصلات على مسافات طوليه لا تقل عن ٢٠ مره قطر السيخ الملحوم .



وصلات اللحام بالتراكب



وصلات اللحام بأستخدام أسياخ اضافيه